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1.0 INTRODUCTION

Approximately 100 template-type offshore platforms have been installed in seismically-active regions of the world's oceans. As new regions with the potential for significant seismic activity are now beginning to be developed, methods are needed to assist in the preliminary design of the structures which will be placed in those regions. In addition, geological studies have identified the potential for significant or increased seismic activity in regions once believed to be far removed from seismic hazard; structures within these regions are in need of assessment for earthquake loads or re-assessment for increased earthquake loads.

Given the increasing importance of seismic considerations for offshore structures, the Marine Technology and Management Group at U.C. Berkeley has initiated a study as part of its Screening Methodologies for Offshore Platforms project to find means of defining and determining the demands an earthquake may impose on an offshore structure. Earthquake demand calculation procedures will be implemented with the ULSLEA platform assessment program which has been developed in preceding phases of the project (Mortazavi, 1996).

This report documents the results of those efforts. A demand calculation procedure utilizing linear elastic response spectrum analysis (RSA) has been implemented in the ULSLEA program for the purposes of calculating load demands on an offshore structure responding in the elastic or near-elastic regions of critical component (deck legs, diagonal braces, foundation piles) load-displacement behavior. Vibration properties are determined from modal analysis of a simple lumped-mass shear-frame fixed-base model of the structure, and then modified to account for foundation and tower-bending flexibility. Loads estimated using this procedure may be directly compared with the existing stress-based capacities currently calculated by the program to evaluate the potential for significant damage to critical components. This load calculation procedure has been verified against the results of detailed 3-D response spectrum analyses of an 8-

leg structure and a 12-leg structure, as well as against both 3-D response spectrum and 3-D time-history analyses of a 4-leg structure. Procedures for performing seismic reliability analyses have also been implemented; mean-value first-order second-moment (MVFOSM) simplifications are used in estimating the statistical distribution of the earthquake load.

In addition to the load demand calculation procedures, several RSA-based approaches to estimating extreme inelastic demands for structures which may possess significant ductility or post ultimate-load strain capacity have been investigated for possible future inclusion into the ULSLEA program. Brief descriptions of these approaches, and the results of several applications to the 4-leg structure studied previously, are contained in Appendix A.

Finally, a procedure for the calculation of peak accelerations for deck-mounted equipment has been implemented into the ULSLEA program. These accelerations may be used to determine forces for the design equipment mountings. A summary of the approach may be found in Appendix B.

This project has been sponsored by ARCO Exploration and Production Technology, Exxon Production Research Company, Mobil Technology Company, Shell Offshore Incorporated, and Unocal Corporation.

2.0 EARTHQUAKE ANALYSIS FOR OFFSHORE STRUCTURES

2.1 Overview

Determining the response of an offshore structure to seismic excitation can be an extremely complex and demanding undertaking. From determining the excitation to apply to the structure, to developing a response model by which the excitation can be translated into demands on the structure, to determining appropriate measures of structural capacity with which to compare the calculated demands, there exist many detailed effects which must be addressed. The major tasks associated with the analysis process are summarized below:

Determining Excitation: The exact excitation to which the structure will be subjected will be unknown, as the rate and duration of energy release from earthquakes is relatively random. Extrapolating the effects of the energy released from the sites of potential seismic activity to the location of the structure under evaluation is a complicated process, and depends on the geological makeup of the region. Effort must be expended to (1) determine these sites of potential activity, (2) characterize in general terms the magnitude and duration of the associated seismic events, and then (3) determine how this translates to ground motions in the vicinity of the structure.

Demand Modeling: Once ground motions in the general vicinity of the structure have been established, the next task is to determine a model by which the motions may be translated into structural demands. For an offshore structure, this will entail formulating not only the mass, stiffness and damping properties of the structure, but also of the foundation (which usually can not be assumed to be rigid) and of the surrounding water. Depending on the level of excitation considered, many of these properties may be non-linear in nature (for example, the yielding of structural members, or strength and stiffness degradation in the foundation), and may require very detailed analytical methods to resolve accurately.

Checking Capacity: Suitable capacity measures must be developed, in order to compare the demands calculated by the analysis. For the case of structure responding in the elastic range, demand may be expressed as force, and capacity in terms of strength. For structures which possess members which are subjected to forces in excess of their yield strengths, demand must be formulated in terms of displacement; hence, strain-based capacity measures will be required. These strain-based capacity measures will rely upon limits set by testing; as they may also need to reflect cyclic degradation and strain-rate effects, dynamic testing is required to set appropriate limits. Along with basic strength and strain capacity, checks must be made of overall stability.

This report is oriented to developing procedures to perform the latter two tasks which are compatible with ULSLEA screening procedures (Mortazavi, 1996); the first task, determining ground motions, is beyond the scope of this research. Readers desiring more information on the first task are referred to Newmark and Rosenblueth (1971).

This section presents background material relevant to the performance of deterministic seismic evaluations of offshore structures. First, current analysis approaches are reviewed, with special attention paid to their ability to account for various effects which may be part of a seismic assessment. Appropriate structural limit states are then discussed, as this will have a bearing not only on the analysis method selected, but also upon the capacity formulation to be used. Finally, the focus of this phase of seismic analysis methodology development is outlined.

2.2 Analysis Methods

The basic purpose of a seismic analysis is to develop transfer functions which convert the earthquake-induced ground motions at the structure's location to loads and displacement demands on the structure. Two basic methods for this process exist: deterministic and stochastic.

For deterministic analysis, the response of the structure to a known ground motion is determined, using either time-history techniques or by use of a response spectrum (a pseudo-static approach). When applying time-history techniques, the equations of motion governing the displacements of the masses of the structure are solved at discrete time intervals for a given time-dependent record of ground acceleration input at the base of the structure. When using a response spectrum, the peak responses of the individual vibration modes of the structure (which are typically found from modal analysis) are estimated by consulting a series of peak responses, either displacement, velocity or acceleration, from a set of single degree-of-freedom systems with varying natural frequencies subjected to a given time history of excitation (the response spectrum). The peak total response of the structure is then estimated by combining the individual modal responses. These deterministic procedures are well-documented in many references on earthquake engineering; the reader is referred to Chopra (1995) if additional detail is desired.

For stochastic analysis, the imposed ground motions are treated as a random process, for which a power spectral density function (a measure of the energy associated with the different frequencies of applied excitation) is determined. From this power spectral density function, spectral densities for the response quantities may be found through the use of frequency-dependent transfer functions. From these spectral densities, root-mean-square values of the desired response quantities may be estimated, along with the variance in these quantities. The reader is referred to Clough and Penzien (1975) and Newmark and Rosenblueth (1971) for a treatment of stochastic analysis procedures.

Of the two methods, the deterministic approaches have wider acceptance as they are easier to understand and apply. Stochastic methods are seldom applied unless there is a strong frequency-dependent character of the excitation; for example, some soil systems possess stiffness and damping which is not invariant with excitation frequency. This research will focus on the development and application of deterministic seismic analysis methods.

In comparing time-history and response spectrum analysis methods, the following points should be noted:

Accuracy of Peak Response Estimates: Time-history analysis will provide time-dependent estimates of member forces and mass displacements; from these the peak total responses of the various structural elements can be directly observed. Response spectrum analysis approximates the peak total responses of elements by combining individual modal responses; the phasing between these responses is not determined and is left to the analyst to judge.

Capturing Non-linear Effects: Time-history analysis will give estimates of member forces and mass displacements for each discrete time step; as the equations of motion governing the system's masses are being solved directly at each time step, the stiffness and damping properties of the structure may possess significant non-linear force-displacement relationships; examples would be elements which allow for changes in strength and stiffness over time. In addition, it is possible to explicitly consider geometric non-linearity which may result from P-Δ effects. When applying response spectrum analysis, it is assumed that the structure possesses linear stiffness and damping: yielding and stiffness degradation within a mode is impossible to consider explicitly, as it is not known when yielding will occur and how the modes will be affected. Instead, several procedures exist for adapting the results of response spectrum analysis to the study of non-linear systems; these procedures have been based upon observations of linear and non-linear single and multiple degree-of-freedom systems (Chopra, 1995). These adaptations tend to be semi-empirical in nature, and hence are approximations to the actual non-linear effects. Several of the more common procedures are reviewed in Appendix A.

Numerous studies have been conducted with the intent of comparing results between time-history analysis and response spectrum analysis, for a variety of linear and non-linear multi-degree-of-freedom systems. In general, for linear systems the two methods are found to agree quite well, although for flexible systems with large numbers of degrees of freedom the results may begin to drift apart (Cruz, Chopra, 1985). For non-linear systems, particularly those with elements which undergo strength or stiffness degradation or have elements with dramatically different strengths, the agreement ranges from excellent to very poor; this is also dependent upon the number of degrees of freedom the structure possesses. A good summary of the limitations of the response spectrum approach for non-linear multi-degree-of-freedom systems is given by Veletsos and Vann (1971).

The decision to apply response spectrum techniques as opposed to time-history techniques is dependent upon the following factors:

Cost of Analysis: Time-history analysis requires a substantial amount of computing capability, which increases exponentially as the detail of the system being modeled increases. Also, the amount of data which is generated and must be processed is extensive. Response spectrum analysis is far less intensive on computer capability and data processing, as it is essentially a static analysis using the peak forces estimated from a series of single degree-of-freedom time-history responses performed earlier.

Importance of Structure: How much information is required, before a judgment as to the satisfactory performance of the structure can be made? If the structure is expected to respond within the elastic or near-elastic range, and has moderate or low consequences of failure, response spectrum analysis is perfectly suited to estimating the forces on the structure. If, however, the structure is expected to undergo significant non-linear deformation, and possesses significant failure consequences, time-history analysis may be needed to develop a detailed picture as to how the structure will respond in the inelastic region, to ensure strain limits (including cyclic effects) are not exceeded and that instability does not exist.

The usual design approach consists of first applying the response spectrum method, in order to size members and develop a general structural configuration. With the use of a smooth response spectrum, the maximum response of a system can usually be bounded quite well with one analysis. Following this, and depending on the importance of the structure, time-history analysis may be performed in order to better determine knowledge of how the structure may respond to several earthquakes. It is necessary to perform evaluations using several time histories, as the true history of excitation will be unknown. In this fashion, response spectrum analysis may be seen to generate baseline demands on a structure, while time-history analysis provides added confidence as to performance estimates.

A final word should be said about verification of these analysis procedures. To date, real-world verification of these procedures is still lacking (Miranda, Bertero, 1994). Additional effort must be made by the use of case studies to determine where current analytical approaches fail, and what effects may be causing them to deviate from reality.

As the intent of performing ULSLEA is to bound demands on a structure in a simple yet accurate manner, response spectrum analysis offers the most practical approach for getting demand estimates. Procedures will need to be evaluated, however, to account for effects such as structural yielding and soil strength and stiffness degradation, if analysis of systems undergoing extreme events is desired.

2.3 Performance and Structural Limit States

The basic purpose in performing a seismic assessment (as in the case of all environmental load assessments for structures) is to ensure that performance limit states specified by the owner and regulatory bodies are not exceeded. Current API guidelines

(API, 1993) suggest the satisfaction of two limit states for offshore structures subjected to seismic activity:

- A serviceability limit state associated with a moderate or so-called "strength-level" earthquake; this event should characterize the extreme seismic event which is expected within the lifetime of the structure. Members in the structure are to remain within code-specified design limit states, which means the structural response will be essentially elastic.
- A no-collapse limit state associated with a rare, intense or so-called "ductility-level" earthquake; this event should characterize the absolute maximum hazard which would be expected in the region. Members in the structure may undergo loading beyond code-specifications, and hence suffer inelastic deformation, so long as the structure satisfies the no-collapse limit state.

When certain robustness criteria are met, and the ratio between the strength-level and ductility-level "intensities" (which may be taken as peak ground acceleration) are less than two, the code states that no explicit ductility analysis is required; this reflects the fact that structures meeting the robustness criteria will be expected to have ductilities (the ratio of total imposed strain to the strain at which a member first exceeds its code value) of at least two.

These limit state requirements have a direct bearing on the selection of the method of analysis to be used. If the focus of an evaluation is on the strength limit state, a pseudo-static linear elastic analysis may be conducted to ensure the forces imposed on the members of the structure do not exceed the required strength states. If, however, the focus is on the ductility limit state, and extreme excursions into the inelastic range are expected, more involved methods of analysis may be required.

ULSLEA determines structural capacities based on ultimate strength evaluations of members and critical components. This is suitable when checking for strength demand;

however, for checking strain demands, a different approach will be needed. This will require use of strain limits which have been established by testing; a review of current test data may be required. Also, the response spectrum approach developed for calculating demands will need to provide the means of finding displacement demands in addition to strength demands.

2.4 Focus of Current Project Phase Seismic Assessment Efforts

The focus of this phase of research into seismic demand calculation for use with ULSLEA will be on determining strength demands. An approach based on response spectrum analysis will be applied, using a modal analysis procedure with appropriate simplifications to ensure the process can be implemented on a personal computer. This procedure will then be verified against the results of more detailed (both modal and time-history) analyses to determine its limitations.

In addition to developing a strength demand calculation approach, effort will also be devoted to finding adaptations to the response spectrum method which may make its use in finding strain demands more viable. Strain capacity measures for critical components will also be studied. The findings of this additional effort are documented in Appendix A.

3.0 A RESPONSE SPECTRUM APPROACH FOR SEISMIC STRENGTH DEMAND CALCULATION

In this section, a response spectrum approach for use determining seismic strength demands on platforms is described. The response spectrum method for multi-degree-of-freedom systems is briefly summarized, followed by a discussion of limitations of the approach. A model suitable for use in capturing the essential characteristics of response for platforms is then proposed, making use of simplifications to keep the level of analysis effort required low. Finally, procedures for determining how many modes to include and how to combine modal responses are described.

3.1 Response Spectrum Analysis for Multi-Degree-of-Freedom Systems

As mentioned in the previous section, response spectrum analysis (RSA) offers the most practical means of determining seismic strength demands for platforms in a cost-effective yet accurate manner. As a background to adapting this approach to the analysis of offshore platforms, the RSA approach for application to multi-degree-of-freedom (MDOF) systems must first be reviewed, and its limitations made clear. What follows is a summary of the RSA approach; readers desiring additional information should consult Chopra (1995).

In applying RSA to the evaluation of a large, complicated structure, an analyst must follow several important steps. First, the vibration properties of the structure (mode shapes, periods and damping ratios) must be determined. This may be done either experimentally (taking actual vibration measurements of the structure in the field), semi-empirically (through application of a code-type estimating procedure such as that contained within the Uniform Building Code), or by developing a numerical model and solving for the properties of free vibration. As the first approach is relatively difficult to perform, and the second approach may involve too many generalities (for example, not accounting for stiffness discontinuities along the height of the structure), numerical modeling offers the most practical means at getting estimates of the vibration properties. A typical numerical model of a platform structure may be seen in Figure 3.1:

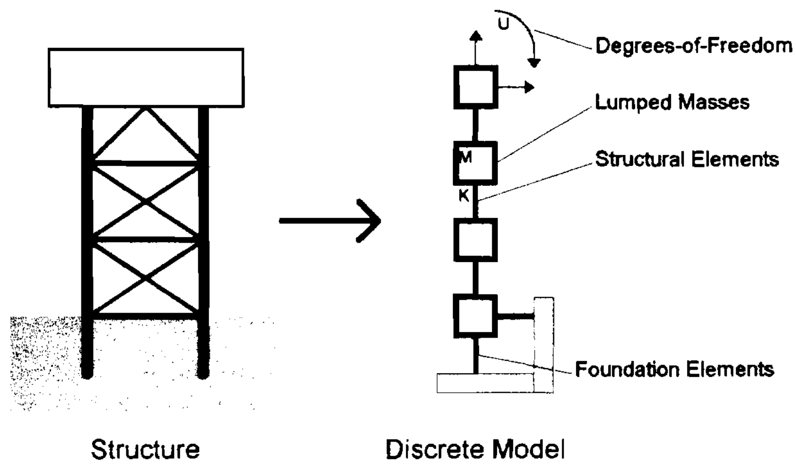

Figure 3.1: Discrete Numerical Model of Structure

Free vibration, neglecting damping, of a multi-degree-of-freedom (MDOF) system is governed by:

$$\mathbf{m}\ddot{\mathbf{u}} + \mathbf{k}\mathbf{u} = \mathbf{0}$$

where: $\mathbf{m}$ = square matrix of lumped masses

$\mathbf{k}$ = square matrix of stiffness properties

$\ddot{\mathbf{u}},\mathbf{u}$ = vectors of acceleration and displacement of each lumped mass

This equation of dynamic equilibrium represents a series of uncoupled differential equations governing the free response of the system. The free vibration properties of the system will be found by the solution of the resulting matrix eigenvalue problem:

$$\mathbf{k}\phi_n = \omega_n^2\ \mathbf{m}\,\phi_n$$

where: ϕ_n = natural shape of vibration for mode n

ω_n = natural frequency of vibration for mode n

This problem is the subject of classical modal analysis, and has been studied extensively over the years for a variety of physical problems. Numerous methods are available for its solution (see Appendix E).

Once the mode shapes and frequencies of the system have been determined, and estimates have been made of the damping ratios ξ_n associated with each mode, the response spectrum appropriate to the location of the structure may be consulted to find the peak responses associated with each mode. A response spectrum is a record of the peak responses (either displacement, velocity, or acceleration) of a group of single-degree-of-freedom (SDOF) systems with various natural periods and values of damping subjected to a time-history of excitation. When developing a seismic response spectrum, this excitation will be a time-history of earthquake excitation. A response spectrum may be developed from the use of a single excitation record, or it may be developed from an ensemble of such records. In the later case, the resulting spectrum is "smoothed" along the overall peak responses irrespective of the exact record; this is referred to as a "design spectrum," and it represents an enveloping of the peak responses which might be expected at the site in question. A typical response spectrum is shown in Figure 3.2.

It should be noted that applying the RSA approach to MDOF systems requires the use of a linear response spectrum, i.e. one that has been developed from the peak responses of SDOF systems possessing linear force-displacement relationships. This is a requirement as the mode shapes and frequencies developed for the MDOF system from free vibration analysis assume linear relationships govern the displacement of each DOF. This limitation will be addressed at the end of this section.

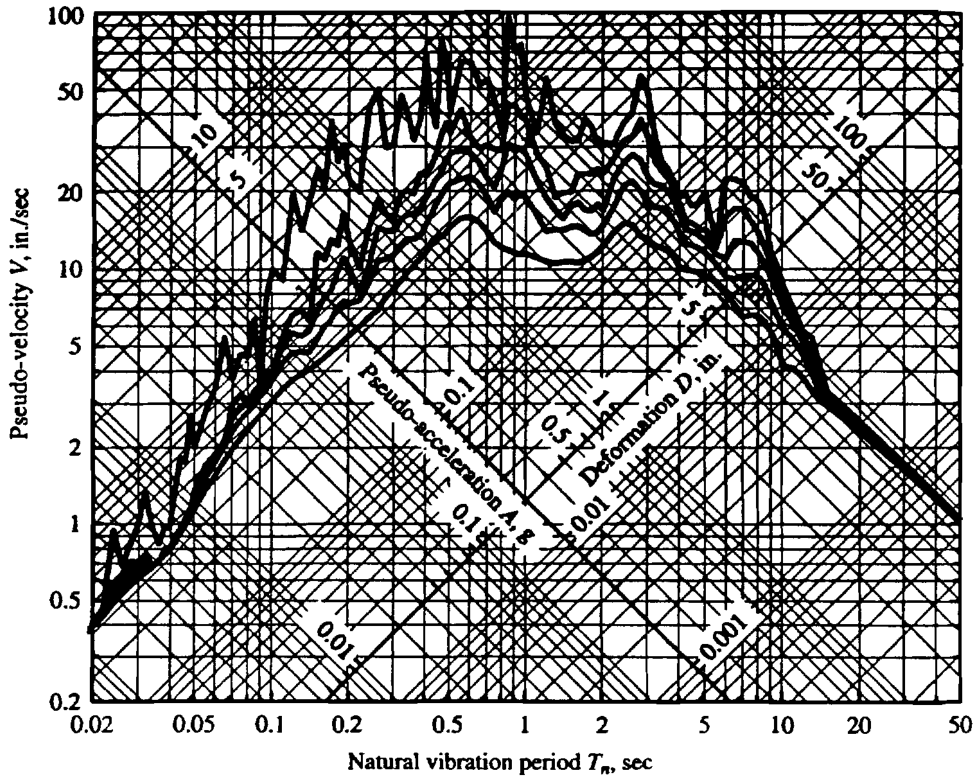

Figure 3.2: Response Spectrum for El Centro, Damping is 0, 2, 5, 10, 20%

From the response spectrum in Figure 3.2, values of peak displacement D_n specific to each natural frequency ω_n can be read. The equivalent static forces associated with each mode for the excitation may then be found from:

$$\mathbf{f}_n = \mathbf{s}_n A_n$$

where: $\mathbf{s}_n = \Gamma_n \mathbf{m} \phi_n$ = distribution of modal inertia forces with: $\Gamma_n = L_n^h / M_n$ = modal participation factor

$$L_n^h = \sum_{j=1}^{N} m_j \phi_{jn}$$

$$M_n = \sum_{j=1}^{N} m_j \phi_{jn}^2 \quad = \text{generalized mass}$$

$A_n = \omega_n^2 D_n$ = pseudo-acceleration; mA is equal to the peak value of the elastic resisting force for a SDOF system

and: n = mode index

j = DOF index

The static forces $\mathbf{f}_n$ may then be used to find member forces and nodal displacements due to each mode using structural analysis. Typical mode-specific responses for the structure modeled as shown in Figure 3.1 would be given by:

$$V_{in} = \sum_{j=1}^{N} \mathbf{f}_{jn} \quad = \text{shear for } i^{th} \text{ level}$$

$$M_{in} = \sum_{j=1}^{N} (h_j - h_i)\mathbf{f}_{jn} = \text{bending moment at } i^{th} \text{ level}$$

$$u_{in} = \Gamma_n D_n \phi_{in} \quad = \text{displacement of node } i$$

To estimate the peak or maximum value of a response quantity, the mode-specific values of the response quantity are first found and then combined. The method of combination is important, as it represents the approximate "phasing" or point in time each peak response occurs relative to every other peak. Various recommendations have been made at to how to combine these responses; most common are the square-root sum of the squares (SRSS), absolute sum (ABS), and complete quadratic combination (CQC). ABS is usually far too conservative and is seldom used. SRSS provides excellent estimates of peak response estimates when the natural frequencies of the individual modes are well-separated. CQC is intended to account for correlation between modes and hence capture effects when modal frequencies are close together. Another proposed combination rule is NRL (Naval Research Laboratory) SRSS; this rule combines the absolute value of the first mode's response with a SRSS of the remaining modes. NRL SRSS is intended to provide better enveloping of the results of response history analysis, as it has been shown that SRSS tends to give response predictions on the low side of response history analysis. The ultimate choice of combination is usually left up to the judgment of the analyst.

Another issue which must be addressed is the number of modes which must be included to capture a given response quantity to the desired degree of accuracy. Some quantities, such as roof displacement, will be dominated by the first one or two modes or a large (5+ DOF) structure, whereas base shear may require substantially more. In many cases, the judgment of the analyst will prevail.

3.2 Non-Linear Effects and Their Inclusion in RSA

RSA for MDOF structures has an important limitation: it is strictly valid for systems possessing constant mass, damping and stiffness; i.e. linear systems. For systems possessing variable mass (such as the changing mass of fluid which interacts with a submerged structure), frequency-dependent damping (such as the damping properties of foundations) and non-linear stiffness (the non-linear force-displacement relationships of either a foundation element or a yielding structural element), approximations must be introduced in order to capture these effects within the framework of modal analysis. Principal sources of non-linearity for offshore structures are listed below:

- Hydrodynamic mass and damping
- Foundation mass, stiffness and damping; strength and stiffness degradation from repeated cycling
- Yielding of structural members; strength and stiffness degradation from repeated cycling

Each of these effects is discussed briefly in the following sections, and current approaches for their inclusion in RSA are described. Where applicable, mention is made of where current approaches are known to deviate from reality.

Hydrodynamic effects of the response of vibrating structures have been studied extensively by both numerical and experimental means (Goyal, Chopra, 1989). While it is generally accepted that for submerged slender members (length to diameter ratios less than ten) hydrodynamic damping (both viscous and radiation) can be neglected, the effects of the mass of fluid both displaced and entrained by the movement of the members can substantially change the vibration characteristics of the structure. The common approach to account for this effect is to assume a certain amount of "added" hydrodynamic mass rides with the members of the structure; this amount is assumed to be constant for the purposes of determining the mass properties of the structure. The amount of mass to include depends upon the size, orientation and depth of the members as well as the manner of excitation (Goyal, Chopra, 1989). For circular members undergoing periodic motion it is generally taken to be equal to the mass of the volume of fluid displaced by the member. It should be noted that recent experience (Bannon, Penzien, 1992) suggests that this approach may over-estimate the actual amount of added mass; additional research is needed in order to verify the validity of this approximation.

Foundation effects may have a strong impact on the seismic response of offshore platforms; typically, these platforms are founded upon somewhat flexible supports which may contribute to isolating the structure from the ground much like many modern base-isolating systems. The issues which must be addressed when including foundation effects are: (1) how much foundation mass to include in the model, (2) how to determine appropriate stiffness and damping values, and (3) how to account for cyclic degradation of strength and stiffness. To explicitly consider the interaction between the structure and its supporting foundation during dynamic action usually requires the application of finite-element methods, in which both the structure and the soil are modeled. In addition, it may be necessary to solve the problem within the frequency domain using Fourier transform techniques, in order to account for the frequency dependence the soil stiffness and damping typically exhibit. This problem is further complicated by yielding and cyclic degradation of the foundation, which can only be solved by explicitly considering the duration of excitation through a time-history approach. In many cases even if great detail is used, the results are not guaranteed to be accurate, as the soil properties themselves are very difficult to determine with any high degree of certainty. Detailed modeling of foundations is still an art very much under development.

Several common approximations exist for including foundation effects on the RSA of a structure; they will be summarized here. Including soil mass in the analysis is usually done in a fashion similar to including hydrodynamic mass. A volume of soil, usually considered to be equal to the volume contained within the pile, is considered to ride with the pile as it moves. The volume of soil is usually taken to extend five to ten pile diameters below the ground. Foundation elements are usually treated as linear elements, neglecting frequency effects, with the stiffnesses being taken as equal to the stiffness of a sample which has undergone cyclic degradation (API, 1993). Similarly, the yield strengths of the elements are also reduced to account for cyclic degradation. Damping effects are including by increasing the modal damping ratios.

For structures which may undergo deformations beyond the elastic range, means must be found to account for the effects of yielding in both the structure and in the foundation, and to find the inelastic deformations imposed upon the structural elements. This has been a subject of intense study over the past 30 years, as it is highly desirable to find adequate means of predicting the inelastic behavior of structures from the analysis of a linear elastic system. Various methods of approximately determining inelastic response from the analysis results of linear systems have been studied over the years; these approaches are examined in Appendix A. The scope of this initial effort will confine itself to structures which do not undergo yielding.

3.3 Response Models for Platforms

With the RSA process summarized, the task now turns to developing a model suitable for determining the important vibration properties of an offshore structure, so that the RSA approach can be applied. This model should allow for the inclusion of the non-linear effects covered by the scope of this phase (hydrodynamics and foundation effects), but should also be amenable to quick solution on a personal computer.

Following previous work performed by Mortazavi (1996), it is assumed the load capacity of a typical offshore platform is governed by the performance of two critical components (see Figure 3.3): bays in the structure (both deck leg and jacket sections) and the foundation.

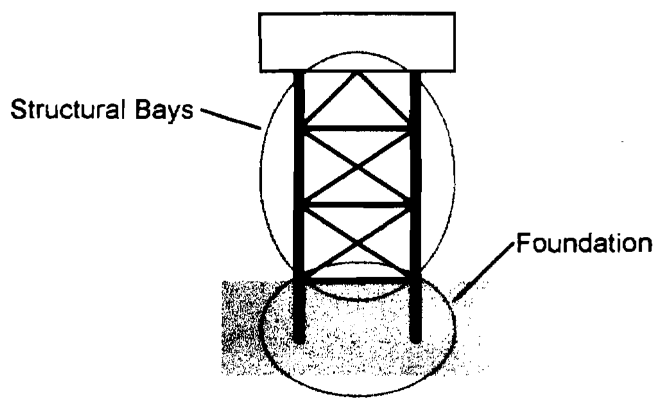

Figure 3.3: Critical Components in a Jacket-Type Platform

A bay in the structure consists of a series of parallel elements (either diagonal braces or unbraced leg sections) which act primarily to resist horizontal loads. The load capacity of a bay is determined by calculating the horizontal load needed to bring the weakest element in the bay to a failure state (either buckling of a brace in a braced section, or hinging of a leg in an unbraced section).

The foundation also consists of a series of parallel elements: the piles (both main and skirt) which support the structure. Foundation capacity is determined by calculating both the axial and lateral load capacity of the individual piles.

The strength demands calculated should therefore be in terms of loads on these critical components. Hence, it will be necessary to estimate horizontal loads (story and base shear demands) on both the structural bays and foundation elements, and vertical loads (axial from both overturning and vertical excitation) on the foundation elements.

3.3.1 Response Directions Considered

To estimate loads in the horizontal and vertical directions, it is necessary to develop a response model or models which capture the performance of the structure in these directions. The bays in the structure will be subject to the greatest load demands when the load direction is parallel with one of the two principal horizontal axes; hence, response models should be developed for each of these directions. Loads on the foundation elements may be found from the loads calculated on the principal axes together with (in the case of axial demand) loads calculated in the vertical direction; therefore, a vertical response model is also needed. If it is further assumed that responses in these three directions are independent of one another (i.e. displacement in one direction does not induce displacement in the other two), then it is possible to develop relatively simple separate response models as shown below in Figure 3.4:

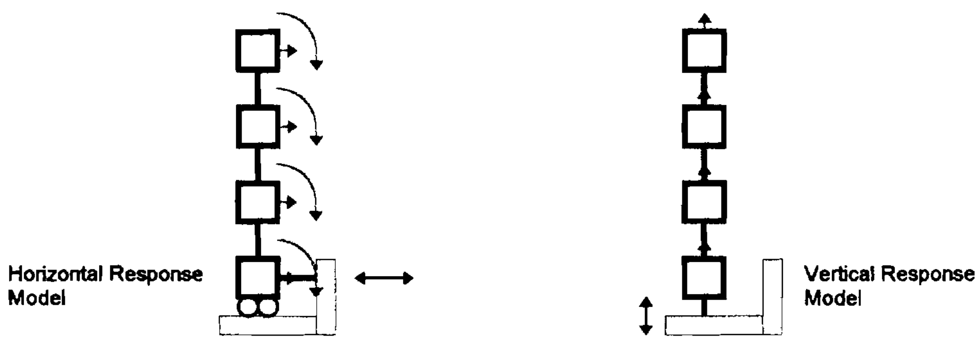

Figure 3.4: Response Models for Horizontal and Vertical Excitation

The scope of this demand modeling will be limited to structures which possess mass and stiffness symmetry on their two principal horizontal axes; hence, lateral-torsional action will not be considered. While symmetric structures may undergo "accidental" torsion due to spatial variations in the applied ground motion, these variations are very difficult to predict, and in most cases accidental torsion results in increases in member forces of less than 4% (Chopra, 1995). Analysts should be aware of this possibility, however, and allow suitable margin for it.

With the demand calculations now reduced to the analysis of three uncoupled response models, it is now desirable to reduce the detail of each model further, in order to simplify the procedures necessary for the modal analysis of each model. However, in the course of simplifying the models, care must be taken to ensure that essential characteristics of response are not lost or distorted.

3.3.2 A Simplified Model for Horizontal Response

The models for horizontal response will be examined first. As shown in Figure 3.4, these are simple lumped mass models (masses lumped as each level or horizontal framing in the structure), with the elements between the DOF representing the combined stiffnesses of the structural members between each horizontal level (i.e. bay or story stiffnesses). The lowest element in the model represents the foundation stiffness.

It would be highly desirable to eliminate the rotational DOF from the structural portion of the model, and to isolate the foundation portion of the model from the structural portion. This would reduce the computational effort to one of solving a matrix eigenvalue problem with diagonal mass and tri-diagonal stiffness matrices (a common formulation for a shear-type building); this type of matrix eigenvalue problem may be readily solved through application of iterative techniques such as the Rayleigh-Stoddola method (Clough, Penzien, 1975). However, these DOF represent significant aspects of structural response, which cannot be neglected. Instead, several simplifying procedures by which the overall effects these DOF can be accounted for will be applied.

Approximating Foundation Flexibility:

Veletsos (1978) has proposed a two-stage procedure by which the period-lengthening effects of foundation flexibility (horizontal as well as rotational) may be accounted for without explicit inclusion in the mass and stiffness formulations used for the modal analysis. Recognizing that foundation effects are concentrated in the first mode responses of a MDOF system, it is possible to first determine the periods and mode shapes of the structure as though it were supported on a fixed base, and then modify the first mode period to account for foundation flexibility effects. In addition, guidelines for adjusting the modal damping ratio to account for foundation sources of damping are also given. It must be noted that the forces acting on the foundation itself must necessarily be approximated, as the foundation mass is not considered in the fixed-base analysis of the structure. This procedure assumes that non-linearity in the soil stiffness and damping will be small, and that cyclic degradation of strength and stiffness may be accounted for by using cyclic test strength and stiffnesses in the model.

The effective fundamental period of a MDOF structure undergoing horizontal excitation can be expressed by:

$$\tilde{T}_1 = T_1\sqrt{1+\frac{k_1^*}{K_x}\left[\frac{1}{1-(T_o/\tilde{T}_1)^2}+\frac{K_x h_1^{*2}}{K_\theta}\right]}$$

where: T_1 = fundamental period of the fixed-base structure

T_o = natural period of foundation mass

$k_1^* = \dfrac{4\pi^2 M_1^*}{T_1^2}$ = effective horizontal stiffness of the fundamental mode of the fixed-base structure

K_x = horizontal stiffness of foundation

K_θ = rotational stiffness of foundation

$h_1^* = \dfrac{L_1^{\vartheta}}{L_1^{h}}$ = effective modal height of fundamental mode of fixed-base structure

$M_1^* = \Gamma_1 L_1^{h}$ = effective modal mass of fundamental mode of fixed-base structure

$L_1^{\theta} = \sum_{j=1}^{N} h_j m_j \phi_{j1}$

The fundamental period of the fixed-base structure is found from modal analysis, whereas the natural period of the foundation mass may be estimated from:

$$T_o = 2\pi \sqrt{\frac{W_o}{g K_x}}$$

where: W_o = weight of foundation mass included in model

g = acceleration due to gravity

The weight of the foundation mass included in the model consists of the weight of any horizontal framing and mud mats on the bottom of the jacket structure, the weight of pile steel in the foundation and the approximate weight of soil entrained with the movement of the jacket and piles.

K_x and K_θ represent pile group stiffnesses, and can be estimated using procedures outlined in Appendix D. The effective damping ratio for the fundamental mode of the structure may be expressed as:

$$\tilde{\xi} = \xi_o + \frac{\xi}{(\tilde{T}_1 / T_1)^3}$$

where: ξ = damping factor of the fundamental mode for the fixed-base structure (usually 2%-5%)

ξ_o = foundation damping, including radiation damping and soil material damping (in the range of 1% to 15%)

To obtain an estimate of the shear imposed on the foundation, the maximum base shear of the fixed-base structure (considering all modes) is combined with an approximate value of the inertia force of the foundation mass:

$$V_o = \frac{W_o A_o}{g} + \sum_{i=1}^{N} V_i$$

where: A_o = pseudo-acceleration of the foundation mass calculated from the response spectrum

V_i = base shear contribution of each individual mode

While the notation above indicates simple summation is used to combine the individual contributions to the foundation load, other rules of combination, such as SRSS or NRL, may be used.

Cantilever Action of the Structure:

The rotational DOF in the structural portion of the model represent the overall bending or cantilever action of the platform. The effect of this action is to lengthen the periods of the first few horizontal vibration modes, and to increase the displacements of the top DOF relative to the lower ones (Figure 3.5).

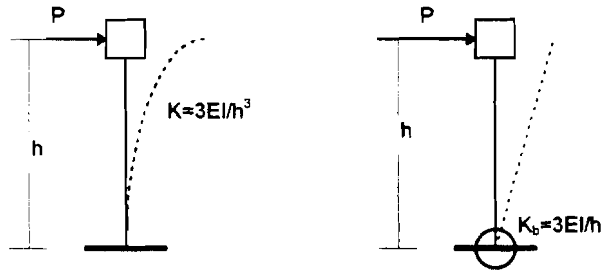

Figure 3.5: Approximating Bending Effects of Structure

It will be assumed that the changes in mode shape are small, and hence will be neglected. The period-lengthening effects of cantilever action can be bounded by modifying the fixed-base response of the structure in a manner similar to that proposed by Veletsos (1978). The bending of the structure is assumed to be similar to the rotation of a rigid structure on a flexible base. To give similar tip displacements for the same tip load, the stiffness of the rotational element may be estimated from:

$$K_b = 3EI/h$$

where: I = moment of inertia of the structure cross-section

h = height of the structure

E = modulus of elasticity

This rotational spring is then considered to act in series with the spring K_θ which represents foundation rotational flexibility.

Additional Effects:

Two additional effects which have not been explicitly included in the model need to be addressed: batter effects and P-Δ effects. For a battered structure, the legs of the structure will contribute to the shear resistance in each story of the structure (increasing in effect towards the bottom of the structure); this will increase the stiffness of the structure and hence lower the period. The magnitude of this effect is unknown (and perhaps impossible to generalize across many configurations); however for the purposes of this study it will be assumed to be small.

P-Δ effects can play an important role in increasing the effective load on a structural bay or story; they also contribute to lengthening the period of the first natural mode of response. Previous research (Gates, et al., 1977) has indicated these changes are small for structures in shallow to medium depths, and hence they will be neglected here as well.

With these simplifications made, the platform may be modeled as a shear frame, with lumped masses (including hydrodynamic mass, discussed in Section 3.3.4) at the levels of horizontal framing. The stiffness of each bay will be approximated by considering only the stiffness contributions of the diagonal braces in each bay (each bay a parallel system of braces) or the stiffness contributions of the jacket legs and piles if there are no braces (again, a parallel system of elements).

3.3.3 A Simplified Model for Vertical Response

The model for vertical response will now be examined. As the main quantity of interest which comes from the analysis of vertical response is the axial load demand on the foundation piles, the model will be reduced in scope so that this quantity can be estimated without much detail. Hence, a model consisting of one DOF is proposed (see Figure 3.6). The mass of the structure, including hydrodynamic mass, is lumped at the DOF; the stiffness element consists of the axial stiffnesses of the piles above the mudline (together with the jacket legs, if the legs are grouted) acting in series with the axial stiffnesses of the piles below the mudline. The hydrodynamic mass will be estimated as described in Section 3.3.4; pile axial stiffnesses will be calculated as described in Section 3.3.2.

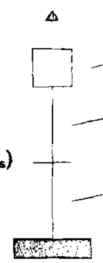

Figure 3.6: Simplified Vertical Response Model (SDOF)

3.3.4 Added Hydrodynamic Mass

The approximate added mass per unit length for cylindrical members undergoing translation is (Newmark, Rosenblueth, 1971):

$$m_{added} = \rho_w \pi r^2 \sin \theta$$

where: ρ_w = density of the surrounding fluid

r = radius of the member

θ = the angle between the cylindrical length axis and the direction of translation

However, it should be noted that the added mass is also dependent upon the proximity of the member to the free surface. Goyal and Chopra (1989) have documented the variation of added mass along the height of circular cylinders of various diameters; it is demonstrated that the added mass begins to drop off rapidly when within 0.1 H_o of the water's surface. Hence, the following approximations are used to scale the amount of added mass included in the weight of the structure:

$$m_{added}(z) = m_{added} \text{ when } z > 0.1\ H_o$$

$$m_{added}(z) = m_{added}(z / 0.1\ H_o) \text{ when } z \leq 0.1\ H_o$$

where: z = depth below the surface

H_o = water depth

This added mass is included in the mass matrices of the structure model when considering both lateral and vertical excitation. It should be noted that the estimation of added mass effects on tower structures subject to seismic excitation is still an area of active research. Recent experience with two offshore structures have indicated the use of current techniques leads to possible overestimation of added mass effects (Bannon, Penzien, 1992). This can have serious consequences on demand estimates, as it leads to inaccuracies in period estimates. Experimental results obtained by Clough (1960) indicate that the added mass associated with a member is strongly dependent on the member's flexibility; Clough (1960) suggests the use of added mass coefficients ranging from 0.6 for flexible members to 1.0 for stiff members in order to account for this dependence.

3.4 Determining Total Strength Demand

With response models suitable for estimating the load demands desired developed, it is now necessary to determine (1) how many modes to include for the case of horizontal excitation when the number of structural bays is large, and (2) how best to combine the individual responses.

Keeping the number of modes considered for horizontal response simplifies the modal analysis procedure by reducing the number of iterations. It is of little use to expend computational time determining modal characteristics for modes which will not contribute significantly to the quantities of interest. A general rule for determining how many modes to include is that of establishing a lower bound of the amount of effective mass (M_n^*) captured in the response. Typically, limits on the minimum amount of participating mass required range from 90% to 95%. Thus, when the total effective mass determined is equal to or above these limits, no additional modes will be calculated.

For the issue of combining modes, a bounding approach may be used. A lower-bound load estimate may be obtained by application of the SRSS modal combination rule, whereas an upper-bound load estimate may be found from applying the ABS combination rule. As an alternative, NRL-SRSS may be used in lieu of ABS, as ABS is extremely conservative.

4.0 VERIFICATION OF SEISMIC STRENGTH DEMAND MODEL

With simplified models for determining vertical and horizontal responses proposed, it is now necessary to verify the accuracy of these models. Verification in this case will take the form of comparing the results of strength demand calculations obtained by application of the simplified models together with RSA to results obtained using more detailed models and analysis methods. The comparisons which have been made are listed below:

1. Comparison of results from simplified response spectrum analysis (SRSA) with results from 3-D detailed RSA of Platform G, an 8-leg jacket structure. Results from application of a modified UBC approach are also shown.
2. Comparison of results from SRSA with results from 3-D detailed RSA of Platform H, a 12-leg jacket structure. Results from application of a modified UBC approach are also shown.
3. Comparison of results from SRSA with 3-D RSA and 3-D time-history (TH) results of the Southern California Test Structure using a flexible foundation. Results from application of a modified UBC approach are also shown.

For each verification case, a brief summary of the structure and analysis method used is given. Comparisons of the strength demands calculated from both the simplified approach and the detailed approach are then shown, along with estimates of natural periods.

4.1 Platform G

Platform G is an 8-leg drilling platform sited in 265 ft of water in San Pedro Bay off Southern California (see Figure 4.1). It was designed to support 80 24 inch-diameter conductors. The platform has two decks located at +45 ft MWL and +64 ft MWL respectively; the deck bay is braced. The jacket is battered 1:7 in the broadside direction, and 1:12 in the end-on direction. The main diagonals range from 20 inch-diameter (w.t. 0.75 inch) to 36 inch-diameter (w.t. 1.125 inches). The corner legs of the jacket are 71 inch-diameter (w.t. 1 to 2 inches), while the interior legs are 54 inch-diameter (w.t. 0.675 to 2 inches); the legs have heavy joint cans but are not grouted. The corner piles of the platform are 66 inch-diameter, and penetrate to 264 ft. The remaining exterior piles are 48 inch-diameter, and penetrate to 232 ft. The soil profile at the site is listed below:

0-12 ft	sandy silt
12-30 ft	medium stiff clay/silt
30-100 ft	sand and gravel
100+ ft	stiff clay/silt

The majority of the structural members are 36 ksi steel, while the piles are 50 ksi steel.

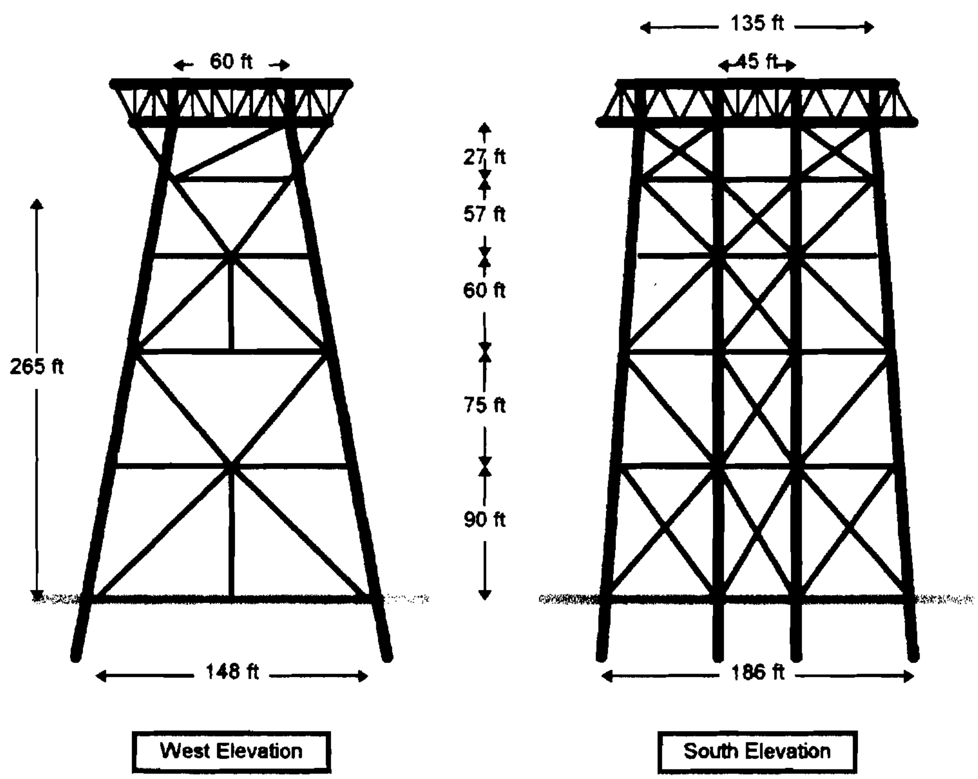

Figure 4.1: Platform G Elevations

A simplified response spectrum analysis of this platform was conducted using the procedures described in Section 3.0. Lumped mass models were developed for the three principal directions of excitation (broadside, end-on, vertical); for the horizontal load models, the deck masses were combined and assumed to be at the location of the lower deck. Foundation stiffnesses were derived according to procedures previously documented by Penzien (see Appendix D); an elastic shear modulus of 2 ksi and poisson's ratio of 0.49 were assumed, and pile axial stiffnesses were assumed to be EA/L. These values were chosen as they gave stiffnesses which were in good agreement with those used in the design. The lumped masses used in the models were taken from the original design report provided by the sponsors; these masses include the effects of added hydrodynamic mass.

The analysis considered three cases of excitation:

- API response spectrum event (zone 4, soil C, PGA of 0.25g) inducing motion in the broadside direction; platform has no marine growth, and combined deck load is 9,048 kips.
- API response spectrum event (zone 4, soil C, PGA of 0.25g) inducing motion in the end-on direction; platform has no marine growth, and combined deck load is 11,448 kips.
- API response spectrum event (zone 4, soil C, PGA of 0.25g) inducing motion in the vertical direction; platform has no marine growth, and combined deck load is 9,048 kips.

Shear demands from the SRSA for the above loading cases are presented in the following subsections, together with design demands provided by the sponsors derived from a detailed 3-D response spectrum analysis and results from application of a modified UBC approach (see Appendix C). The design demands were determined using the same API spectrum; NRL-SRSS was used to combine modal responses, and linear elastic pile head springs developed from detailed analysis of individual piles were used.

4.1.1 Broadside

Shear demands for the case of broadside excitation are shown in Figure 4.2 for the SRSA, design analysis and modified UBC analysis. Examining Figure 4.2, SRSA with SRSS tends to under-predict response relative to the design calculation (in the range of 2% to 11%), while using NRL-SRSS over-predicts (differences range from 8% to 32%). The modified UBC approach ranges from 3% low to 12% high relative to the design loads. The fundamental period estimated using SRSA is 6% lower than that estimated by the design calculations.

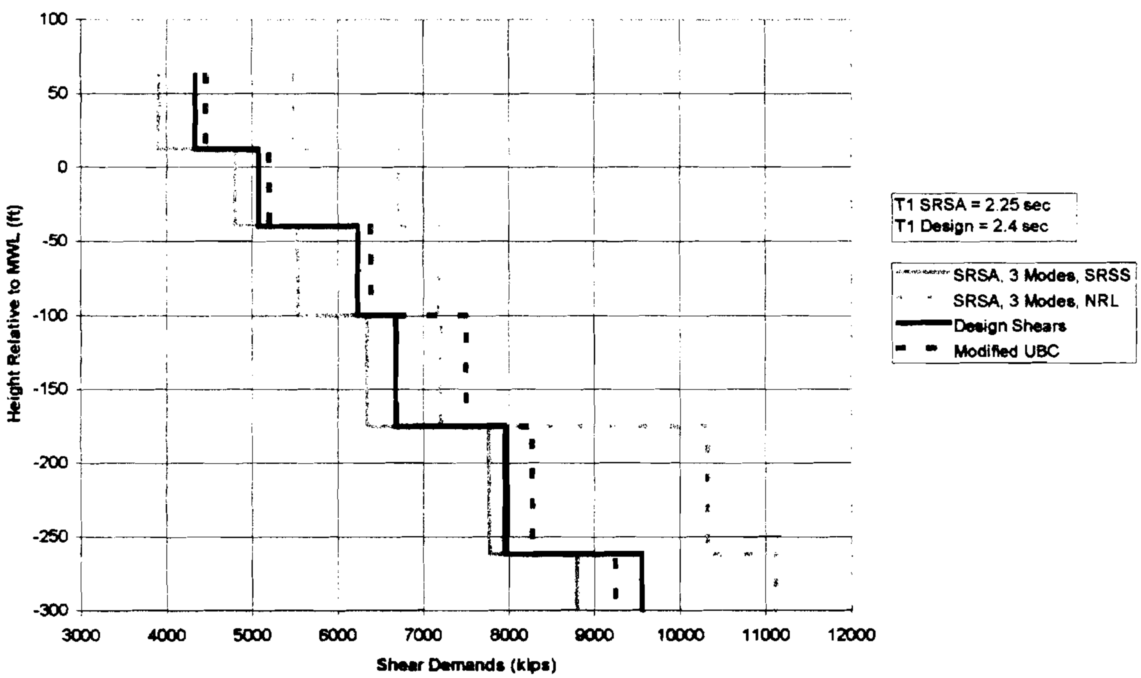

Figure 4.2: Broadside Shear Demands

Figure 4.3 shows the relative importance of the different modal contributions calculated from the SRSA for the broadside case. Examining Figure 4.3, it is clear that response is dominated by the first two modes. Also, it is seen that the 2nd mode period estimated by SRSA is 47% of that estimated during design; this equates to roughly a 20% greater estimate of forces acting in the second mode. This increase, combined with a 5% high estimate of 1st mode forces, accounts for the differences seen between the design analysis and the simplified analysis using NRL-SRSS.

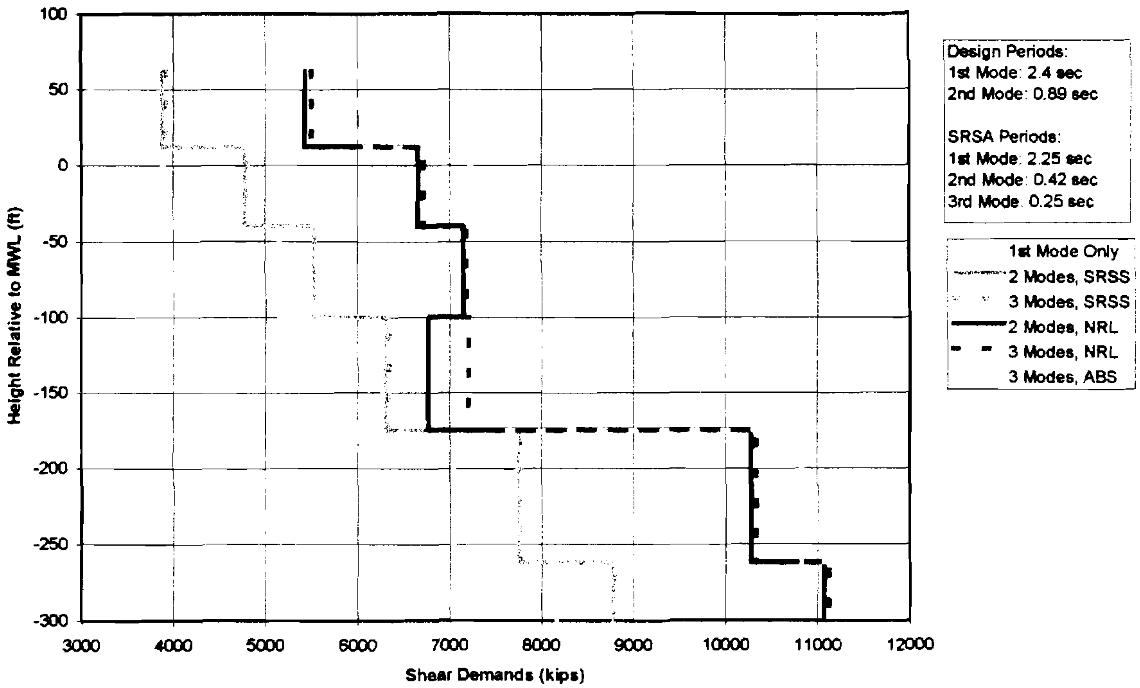

Figure 4.3: SRSA Modal Contributions for Broadside Shear Demands

4.1.2 End-On

Shear demands for the case of end-on excitation are shown in Figure 4.4 for the SRSA, design analysis and modified UBC analysis. For the case of end-on loading, SRSA using SRSS under-predicts response by between 6% to 17% relative to the design calculations, while using NRL-SRSS over-predicts response by between 6% to 30%. The modified UBC approach provides estimates which range from 7% low to 7% high relative to the design values. The fundamental period estimated using SRSA is 6% lower than the period estimated by the design calculations.

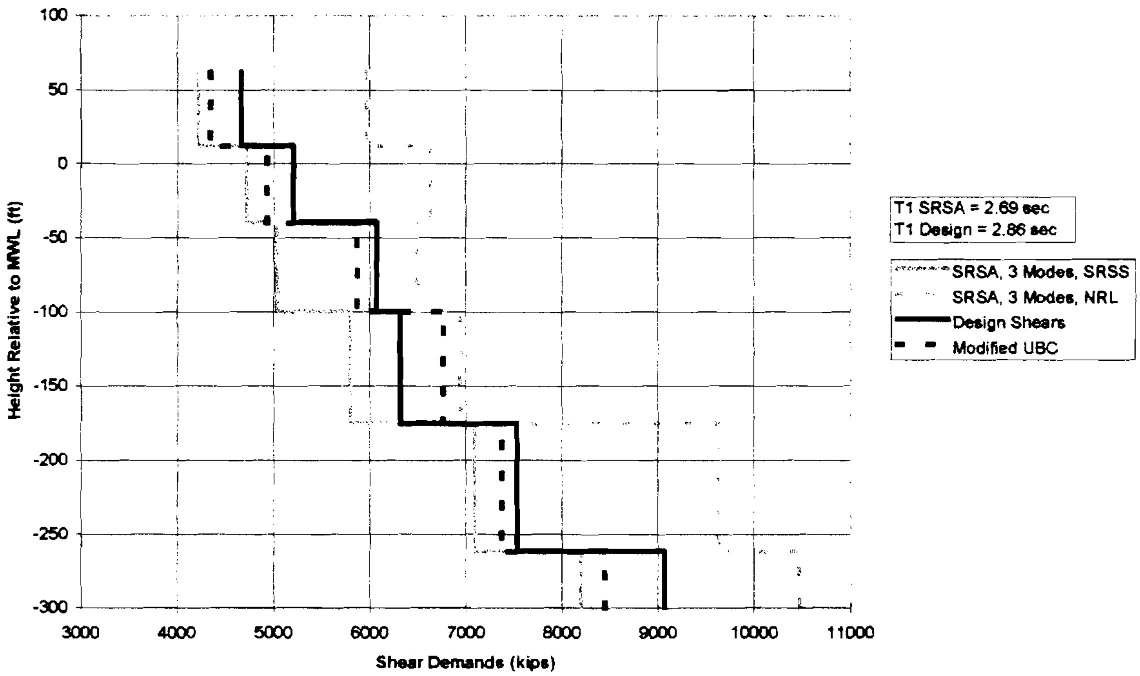

Figure 4.4: End-On Shear Demands

Figure 4.5 shows the relative importance of the different modal contributions calculated from the SRSA for the end-on case. Examining Figure 4.5, it is clear that response is dominated by the first two modes. Also, it is seen that the 2^{nd} mode period estimated by SRSA is 50% of that estimated during design; this equates to roughly a 20% greater estimate of forces acting in the second mode. This increase, combined with a 6% high estimate of 1^{st} mode forces, accounts for the differences seen between the design analysis and the simplified analysis using NRL-SRSS.

4.1.3 Vertical

For the case of vertical loading, the peak vertical force estimated by application of SRSA was 17,160 kips; this is 8% higher than the design value of 15,780 kips. The estimated fundamental period matched the design fundamental period (0.507 sec).

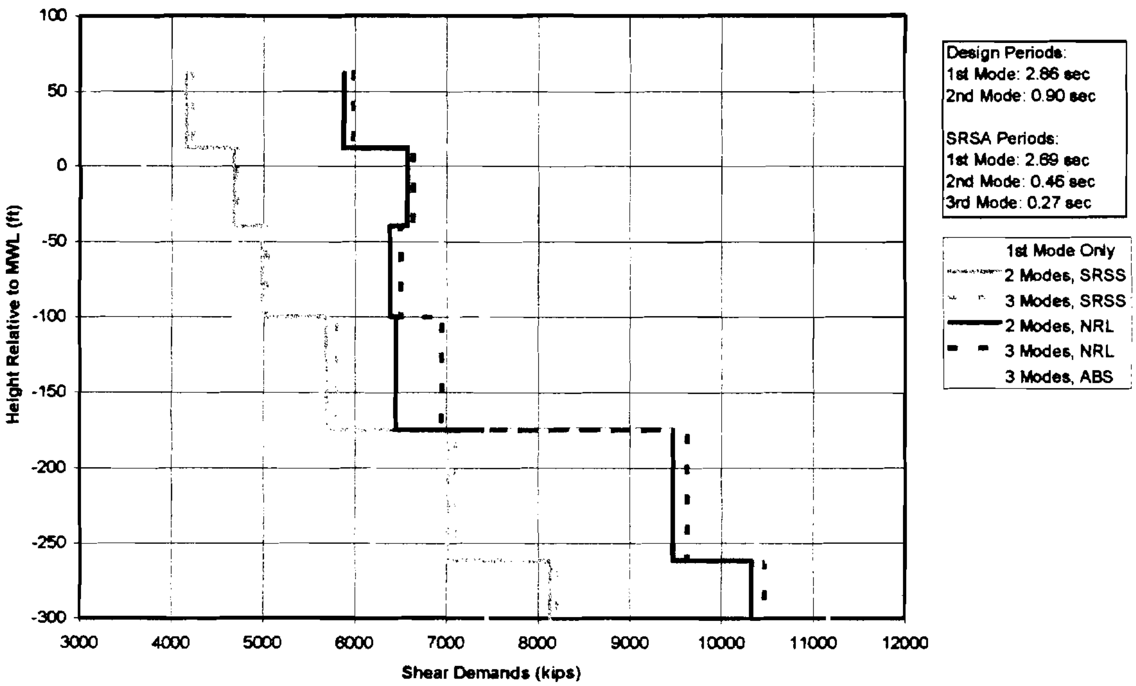

Figure 4.5: SRSA Modal Contributions for End-On Shear Demands

4.1.4 Discussion

Reviewing the results shown in the previous subsections, it can be seen that the demands estimated using the simplified response spectrum model bound the design demands. The SRSA NRL-SRSS demands are significantly higher than the design demands towards the top and bottom portions of the structure; this is due to the fact that the modes beyond the first estimated by SRSA have significantly shorter periods than those estimated during the design effort, and hence are higher up on the response spectrum. The first mode period estimated using the simplified model compares very well with the design period. It is interesting to note that the shear demands estimated using the modified UBC approach compare well with the design shears; as the structure has no stiffness discontinuities, the code forces work well.

4.2 Platform H

Platform H is a 12-leg production platform sited in 257 ft of water in San Pedro Bay off Southern California (see Figure 4.6). The platform has two decks located at +45 ft MWL and +71 ft MWL respectively; the deck bay is braced. The end-on frames of the jacket are battered 1:10, while the exterior broadside frames are battered 1:14. The main diagonals range from 24 inch-diameter (w.t. 0.625 inch) to 36 inch-diameter (w.t. 0.75 inches). The exterior legs of the jacket are 53 to 54 inch-diameter (w.t. 0.75 to 1 inch), while the two interior legs are 47 inch-diameter (w.t. 0.675 to 1 inch); the legs have heavy joint cans but are not grouted. The exterior piles of the platform are 48 inch-diameter; the corners penetrate to 252 ft, while the remainder penetrate to 221 ft. The interior piles are 42 inch-diameter, and penetrate to 200 ft. The soil profile at the site is the same as for Platform G. The majority of the structural members are 36 ksi steel, while the piles are 42 ksi steel.

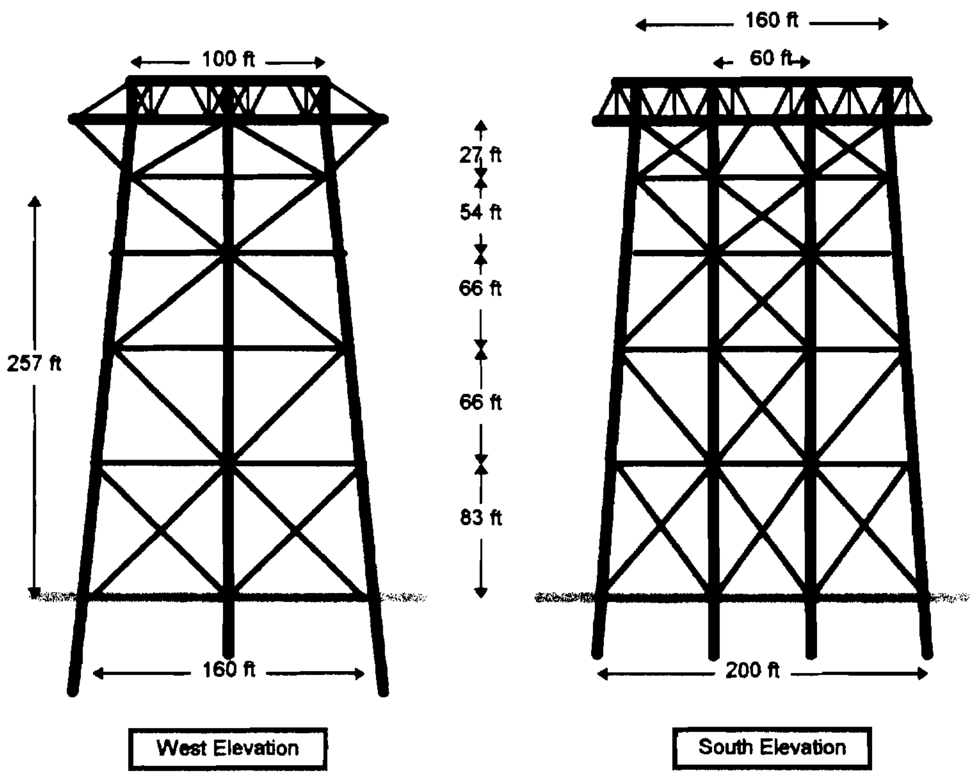

Figure 4.6: Platform H Elevations

A simplified response spectrum analysis of this platform was conducted using the procedures described in Section 3.0. Lumped mass models were developed for the three principal directions of excitation (broadside, end-on, vertical); for the horizontal load models, the deck masses were combined and assumed to be at the location of the lower deck. Foundation stiffnesses were derived according to procedures previously documented by Penzien (see Appendix D); an elastic shear modulus of 2 ksi and poisson's ratio of 0.49 were assumed, and pile axial stiffnesses were assumed to be EA/L. These values were chosen as they gave stiffnesses which were in good agreement with those used in the design. The lumped masses used in the models were taken from the original design report provided by the sponsors; these masses include the effects of added hydrodynamic mass.

The analysis considered three cases of excitation:

- API response spectrum event (zone 4, soil C, PGA of 0.25g) inducing motion in the broadside direction; platform has no marine growth.
- API response spectrum event (zone 4, soil C, PGA of 0.25g) inducing motion in the end-on direction; platform has no marine growth.
- API response spectrum event (zone 4, soil C, PGA of 0.25g) inducing motion in the vertical direction; platform has no marine growth.

Shear demands from the SRSA for the above loading cases are presented in the following subsections, together with design demands provided by the sponsors derived from a detailed 3-D response spectrum analysis and results from application of a modified UBC approach (see Appendix C). The design demands were determined using the same API spectrum; NRL-SRSS was used to combine modal responses, and linear elastic pile head springs developed from detailed analysis of individual piles were used.

4.2.1 Broadside

Shear demands for the case of broadside excitation are shown in Figure 4.7 for the SRSA, design analysis and modified UBC analysis. For the case of broadside loading, SRSA with SRSS tends to under-predict response relative to the design calculation (2% high to 12% low), while using NRL-SRSS over-predicts (differences range from 11% to 29%). The modified UBC approach ranges from 2% low to 13% high relative to the design loads. The fundamental period estimated using SRSA is 5% lower than that estimated by the design calculations.

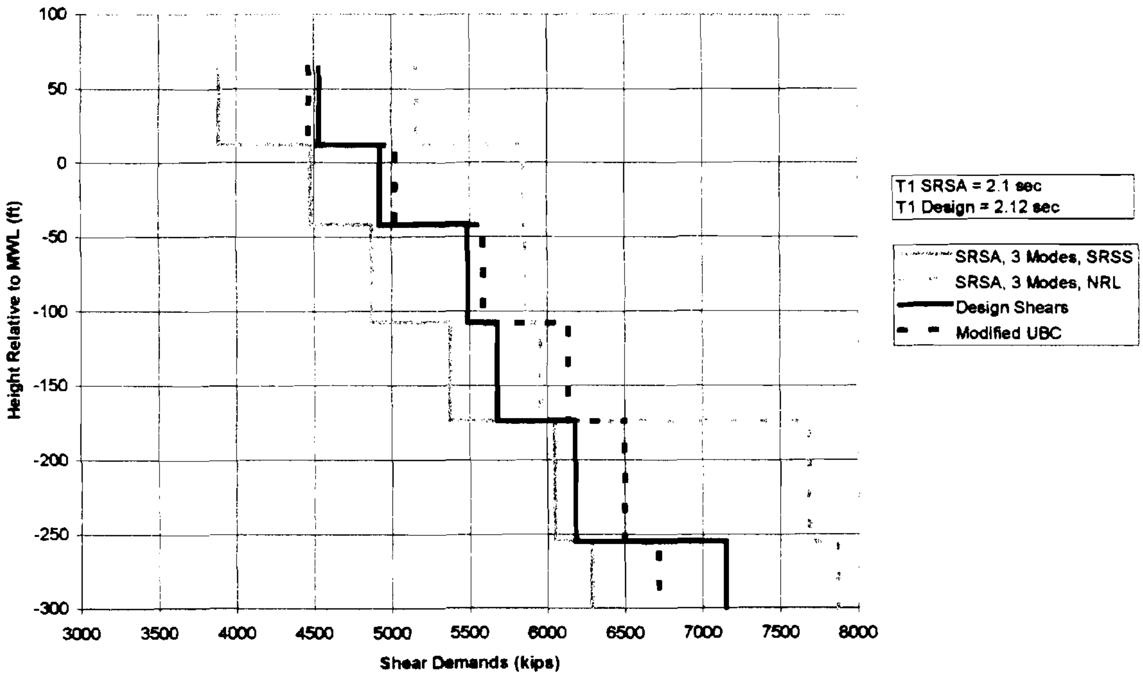

Figure 4.7: Broadside Shear Demands

Figure 4.8 shows the relative importance of the different modal contributions calculated from the SRSA for the broadside case. Examining Figure 4.8, it is clear that response is dominated by the first two modes. The 2nd mode period from the design was unavailable; however, it may be assumed that SRSA has over-predicted it by a similar margin as observed for Platform G. This most likely accounts for the differences seen between the design analysis and the simplified analysis using NRL-SRSS.

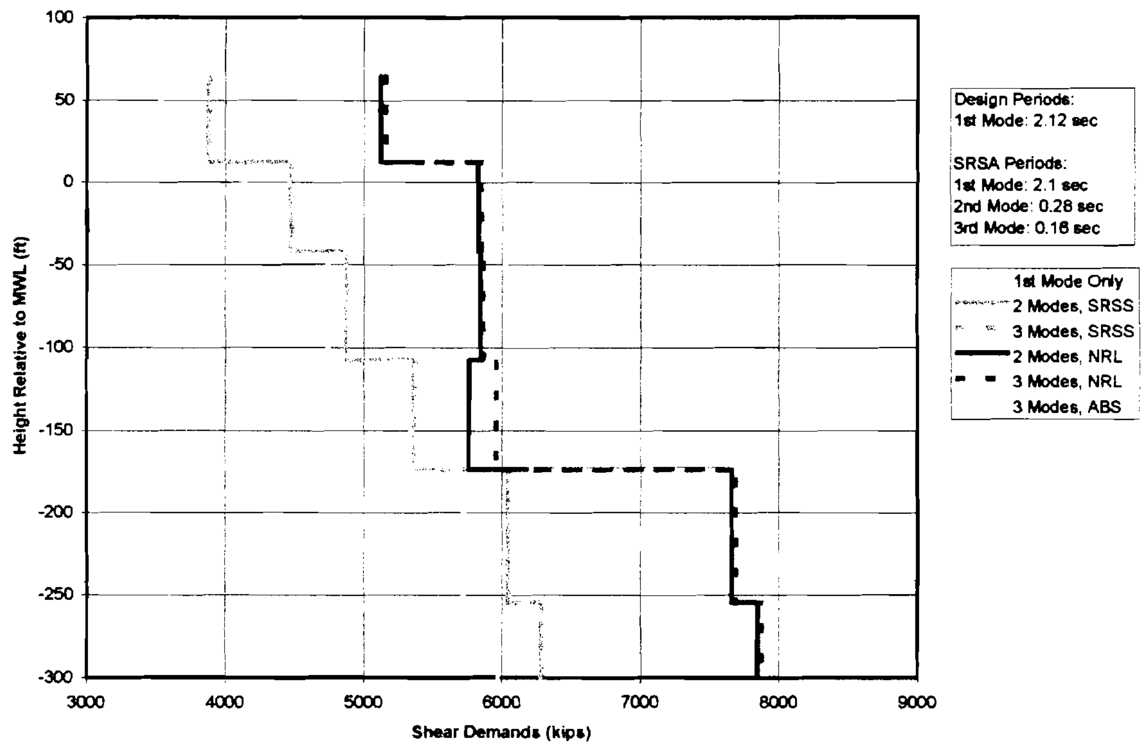

Figure 4.8: SRSA Modal Contributions for Broadside Shear Demands

4.2.2 End-On

Shear demands for the case of end-on excitation are shown in Figure 4.9 for the SRSA, design analysis and modified UBC analysis. For the case of end-on loading, SRSA using SRSS bounds response by between 4% low to 14% high relative to the design calculations, while using NRL-SRSS over-predicts response by between 15% to 43%. The modified UBC approach provides estimates which range from 5% to 24% high relative to the design values. The fundamental period estimated using SRSA is 11% lower than the period estimated by the design calculations. This difference in period equates to roughly a 11% increase in the forces estimated by the simplified methods over those estimated by the design.

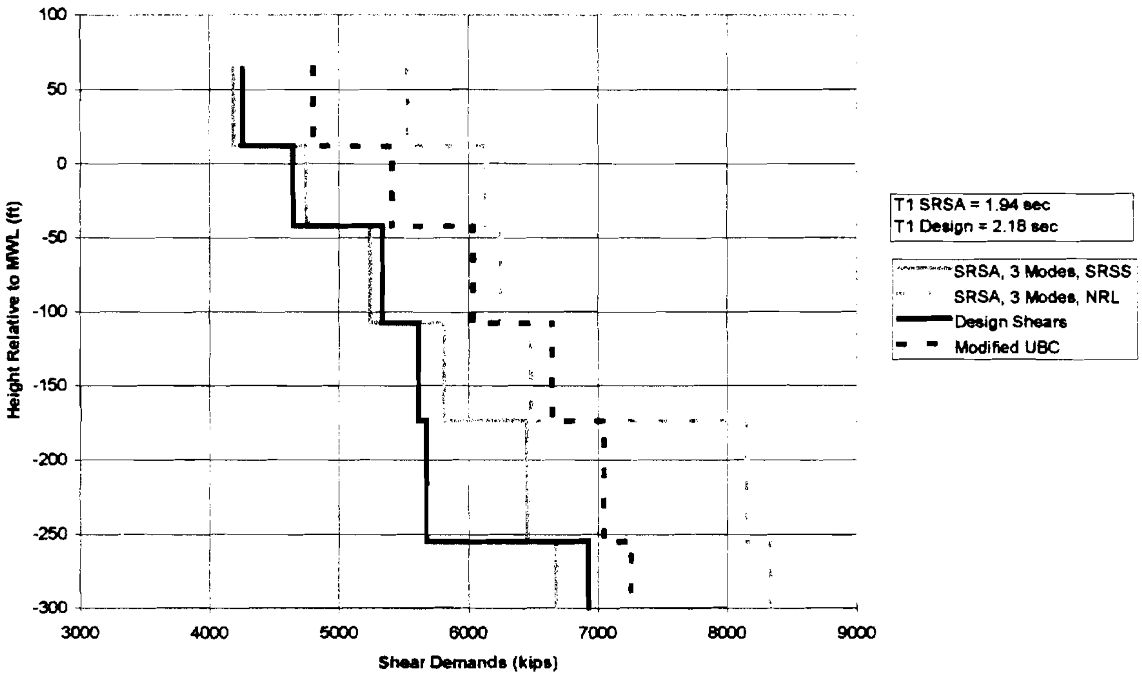

Figure 4.9: End-On Shear Demands

Figure 4.10 shows the relative importance of the different modal contributions calculated from the SRSA for the end-on case. Examining Figure 4.10, it is clear that response is dominated by the first two modes. The 2nd mode period from the design was unavailable; however, it may be assumed that SRSA has over-predicted it by a similar margin as observed for Platform G. This, together with the low 1st mode period estimate, most accounts for the differences seen between the design analysis and the simplified analysis using NRL-SRSS.

4.2.3 Vertical

For the case of vertical loading, the peak vertical force estimated by application of SRSA was 18,996 kips; this is 1% higher than the design value of 18,843 kips. The estimated fundamental period, 0.513 sec, is 24% higher than the design fundamental period. However, it should be noted the design estimated two vertical modes with very close periods, which had 60% and 33% mass participation. The single vertical mode approximation used in this analysis appears to bound the design results quite well.

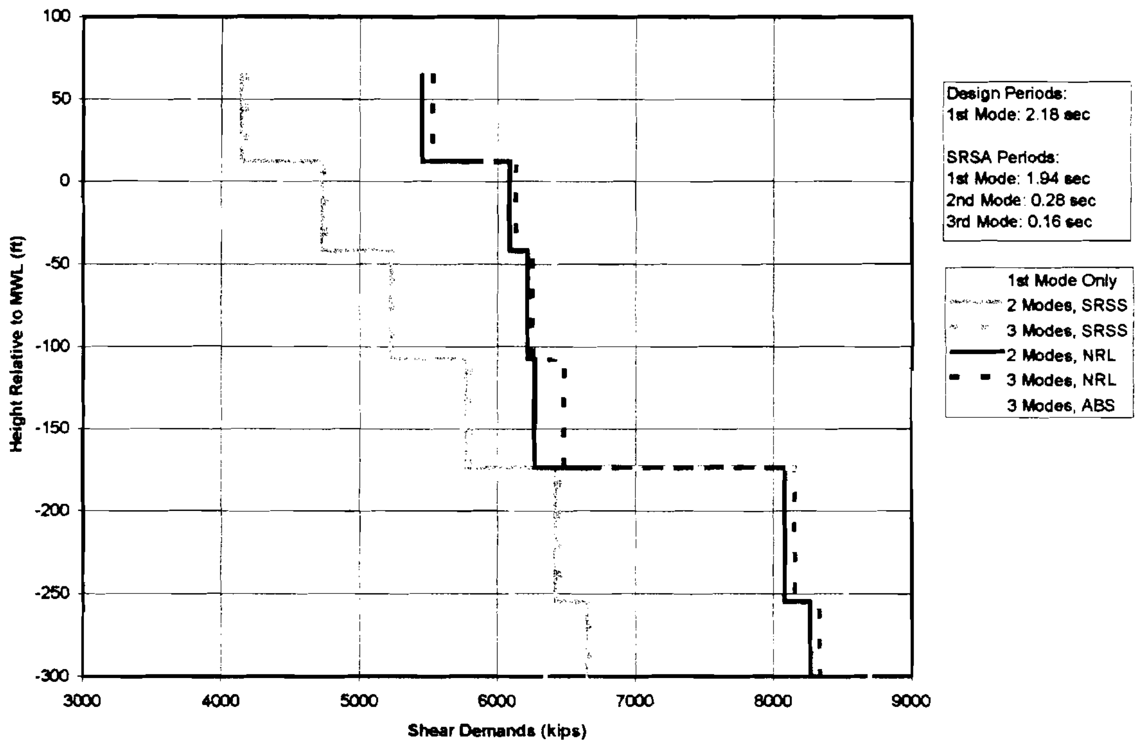

Figure 4.10: SRSA Modal Contributions to End-On Shear Demands

4.2.4 Discussion

Reviewing the results shown in the previous subsections, it can be seen that the demands estimated using the simplified response spectrum model bound the design demands for the case of broadside excitation, but over-predict the demands relative to the end-on design demands. This is due to the short fundamental period estimated for the end-on direction; the corresponding response spectrum acceleration is 11% higher than the design fundamental period acceleration value. The reason for the low period estimate is unclear. Both the end-on and broadside fixed-base periods for this structure were approximately 1 sec; as the broadside foundation rotational stiffness and equivalent tower bending stiffnesses as determined through application of the simplified procedures are lower than that for the end-on direction, the broadside period is longer. One possible source of additional flexibility in the end-on direction was thought to be the bracing between the two decks; the bracing for broadside loads is twice as stiff as the bracing for end-on loads at this level. However, a two-deck model was constructed including this bracing; after analysis it was found there were no significant changes in the vibration characteristics. The possibility that the broadside foundation stiffnesses were under-estimated was also investigated; the initial rotational stiffness did not include the rotational stiffness contributions of the center piles. These were evaluated, however, and were found to be insignificant relative to the overall rotational stiffness. The broadside case points out the importance of getting good period estimates; errors in this estimate can lead to significant over-prediction of load.

It should also be noted that the code forces work well in estimating the demands for the broadside case; the higher demands estimated for the end-on case again reflect the differences in period estimated between the design and simplified models.

4.3 Southern California Test Structure

The Southern California Test Structure is a hypothetical design for a symmetric 4-leg production platform (see Figure 4.11). The design was developed during the late 1970's; scale models of the design were used for cyclic structural tests at U.C Berkeley (Zayas, et al., 1980A,B). The structure is designed for 100 ft water depth. The deck is at +50 ft MWL and supports a load of 5,000 kips. The main diagonals in the first jacket bay are 24 inch-diameter (w.t. 1 inch in top portions and 0.5 inch in bottom portions), while those in the second bay are 30 inch-diameter (w.t. 0.625 inch); the diagonals in the deck bay are 36 inch-diameter (w.t. 0.75 inch). The legs are 78 inch-diameter (w.t. 0.875 to 1.125 inches); they are grouted, and possess heavy joint cans. The piles are 72 inch-diameter (w.t. 1 to 1.5 inches), and are designed for 150 ft penetration in medium to stiff clay. The main structure is A36 steel.

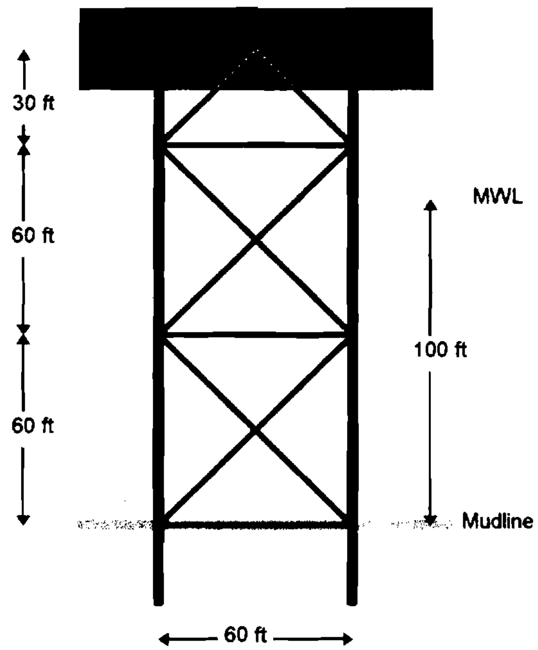

Figure 4.11: Southern California Test Structure

This design has been the subject of numerous studies, primarily because of the data available from the testing of the scale models. Two strength-level analyses have been documented by Gates, et al. (1977); the results of these analyses will be used for comparison purposes against results obtained through application of the SRSA approach and the modified UBC approach (Appendix C). The first of these analyses is a 3-D response spectrum analysis performed using the API zone 4 soil condition B spectrum (PGA of 0.25g). The 3-D model included all members above the mudline, while foundation behavior was represented by linear springs. The second analysis is a time-history study of platform response; the same model that was used in the 3-D RSA was used for this study. The model was analyzed for three different earthquake ground motion records: El Centro NS, Taft S69E, Olympia EW; each of these records was scaled to a PGA of 0.25g.

Both a simplified response spectrum analysis and a modified UBC analysis were also performed for this platform to estimate horizontal load demands. For both analyses, the lumped masses and foundation spring vertical and horizontal stiffnesses listed as being used by Gates, et al. (1977) were used in these analyses (using the Penzien approach in Appendix D, this corresponds to an elastic shear modulus of 2.5 ksi and poisson's ratio of 0.49, and pile axial stiffnesses of 0.9EA/L); the same API response spectrum was also used.

The results of both the Gates, et al. (1977) analyses and the simplified analyses are shown in Figure 4.12. Modal contributions from the SRSA are shown in Figure 4.13.

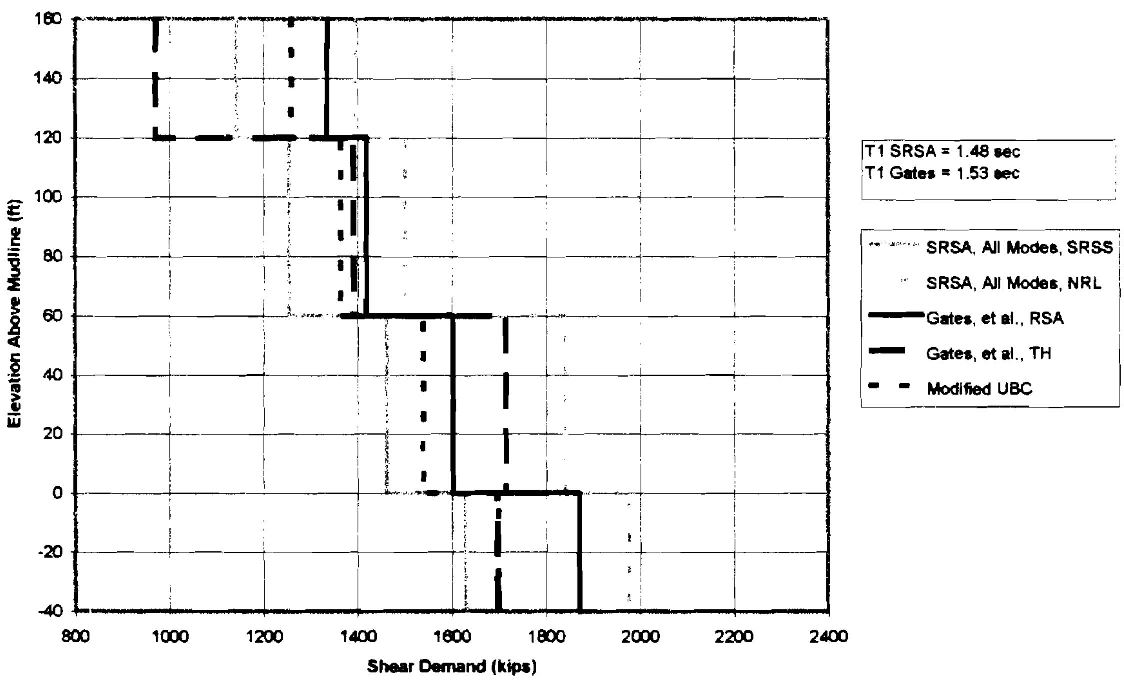

Figure 4.12: Shear Demands

As can be seen from examination of Figure 4.12, both the SRSA (SRSS) and modified UBC approaches under-predict the horizontal shears on the platform by at most 13% and 10%, respectively, relative to the results of the response spectrum analysis performed by Gates, et al. (1977). The SRSA (NRL-SRSS) over-predicts the horizontal shears relative to the Gates, et al. (1977) response spectrum analysis by at most 15%. It should be noted that both the SRSA (SRSS) and modified UBC results parallel the Gates, et al. (1977) response spectrum analysis results quite strongly; possible reasons for the discrepancies may be traced to two sources: the modal combination rule used by Gates, et al. (1977), which is not specified, and the lumped masses used. There is a discrepancy between the total inertial mass above the mudline (9,400 kips) and the actual sum of the lumped masses above the mudline (8,644 kips) listed by Gates, et al. (1977); the lumped masses shown in the report are identified as being part of a detailed model used for a non-linear time-history analysis, and therefore may not be the same as those used for the linear response spectrum and time-history analyses. The difference between these mass values, 8%, would roughly account for the differences observed between the period and shear estimates. More importantly, however, it should be noted that only the SRSA (NRL-SRSS) results envelope the maximum shears developed by the Gates, et al. (1977) time history analysis.

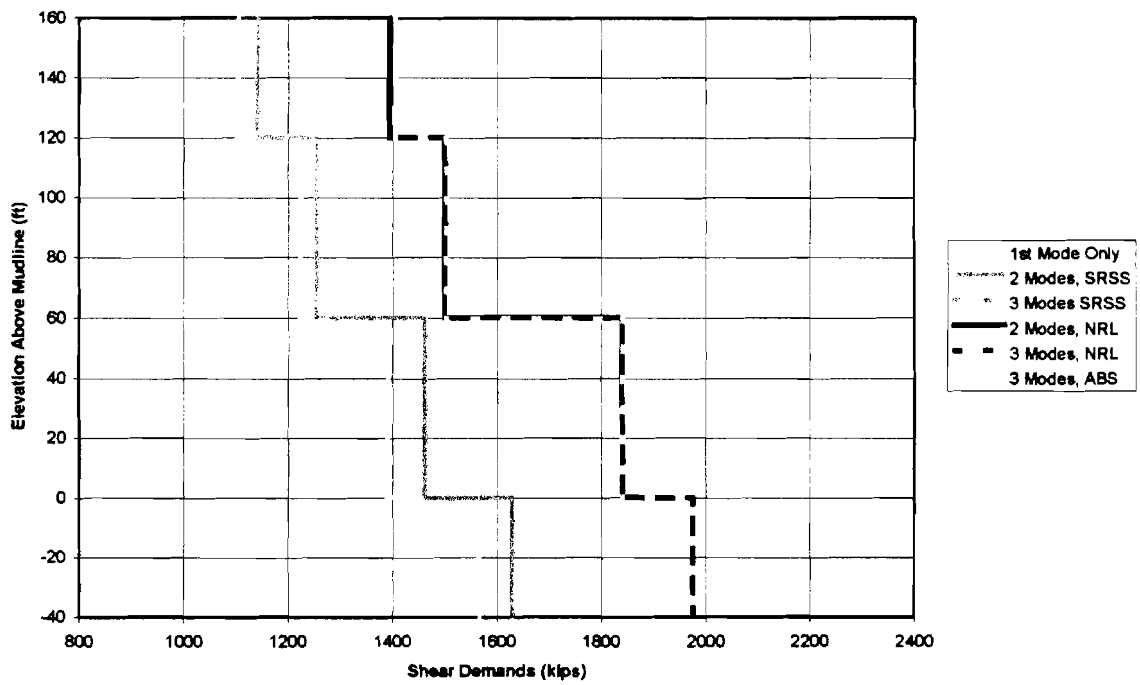

Figure 4.13: Modal Contributions to Response

4.4 Conclusions

Overall, the simplified models capture the vibration characteristics of the platforms quite well, with the exception that higher mode effects tend to be exaggerated. It should be noted that the period estimates are quite sensitive to pile axial stiffnesses and horizontal stiffnesses; this effect has been explored in Appendix D. Additional effort is needed to better define simplified estimates of these quantities, as they are critical to the performance of an accurate evaluation. Of equal importance will be the soil parameters used by these simple formulations; if good soil data is lacking, there will be serious errors in the foundation stiffnesses.

The load to which the platform is subject is quite sensitive to the fundamental period estimates, considering the region in the response spectrum most of these structures tend to fall in. While this may be of less concern for a smooth spectrum, if a jagged spectrum is used there may be substantial error in the estimated load. While this is of little concern for design and assessment purposes (which should use smooth spectra), it may make the procedure of little value for post-event evaluations, where a specific response spectrum may be available.

It should also be noted that the platforms evaluated in these initial cases are in shallow to moderate water depths. For deeper platforms, the effects of tower bending on the mode shapes (in particular, the displacements of the top portion of the structure) may become quite pronounced, and hence may need to be taken into account.

The modified UBC approximation works well when compared to response spectrum results using NRL-SRSS, although it is prone to errors in a similar manner as the SRSA approach due to the procedures used to estimate the fundamental period. This approach involves much less computation relative to performing a modal analysis if period estimating is done by Rayleigh's method or another simplified method. It was believed initially that this approach would lose accuracy, however, for cases in which there are stiffness discontinuities in the structure such as an unbraced deck bay. This issue has been explored in Appendix C; for the cases considered, the error relative to results from modal analysis are not great. The modified UBC approach may be a viable alternative to seismic load computation if modal analysis procedures are not available.

It has been assumed in these evaluations that the added mass approximations commonly used in design are valid. The only means of verifying this assumption is to perform calibration against physical tests and real structures; this issue should be addressed in the future.

Lastly, effort must be concentrated upon evaluating inelastic response, as this is critical to establishing whether the collapse limit state has been exceeded. Appendix A summarizes several approaches to estimating inelastic demands; additional work is needed to test these approaches and determine the limits of their accuracy and applicability.

5.0 SEISMIC RELIABILITY ANALYSIS

A major focus of this research effort is to develop a reliability-based procedure by which the uncertainties associated with the earthquake demands and platform capacities can be accounted for when making a performance assessment. Previous work in the area of reliability analysis using simple systems has been performed by Mortazavi (1996) for platforms subjected to storms; in this previous effort mean-value first-order second-moment (MVFOSM) approximations were used to define distributions for platform component capacities and distributions for demands from storms on those components. Simple models were developed to establish the relationships between the random variables controlling the capacities and demands.

The approach used by Mortazavi (1996) lends itself to adaptation for reliability assessments of offshore platforms subjected to earthquakes. Adapting the procedure for strength-level reliability assessments posses no great effort; however, applying the procedure for ductility-level assessments may prove challenging, given the complex relationship between earthquake motion and ductility demand for a given structure. This phase of the research effort will concentrate on adapting this procedure for strength-level assessments.

In the following sections, basic approaches for assessing seismic reliability for given levels of earthquake will first be reviewed. Next, a simplified distribution for earthquake forces will be proposed.

5.1 Reliability Analysis Approaches

Reliability analysis is a necessary part of any seismic assessment. There exists great variation in the magnitude, duration and rate of energy release of all earthquakes; this uncertainty translates directly into uncertainty in the demands imposed on a structure subjected to an earthquake. Determining the demands is further complicated by the uncertainties existing in the structure's response characteristics, i.e. in the periods and mode shapes of vibration, and energy dissipating mechanisms such as damping.

Current seismic hazard methodology dictates the evaluation of a structure for two levels of hazard: (1) a moderate earthquake, during which the structure should suffer no serious damage, and (2) a severe earthquake, during which the structure should not collapse. The moderate earthquake is chosen as likely to occur on nearby faults within the lifetime of the structure, whereas the severe earthquake is selected based on the maximum earthquake potential of nearby faults. Probabilities of occurrence for the different levels of earthquake at a given location are typically very difficult to define, given the lack of accurate historical ground motion data.

It is not the intent of this research effort to develop probabilistic estimates of earthquake occurrences; readers desiring further information on the subject are referred to Idriss (1985). Instead, the focus shall concentrate on methods of evaluating structural reliability for specific levels of earthquake excitation.

A seismic hazard analysis for a given level of earthquake could be performed in two ways: (1) using multiple time-history analyses, and (2) using response spectrum methods. For the case of using time-history analysis, various earthquake time histories taken as representative of the desired level of earthquake would be used to excite a model of the structure. For each analysis, the fact that a specific level of load or damage had or had not been exceeded would be recorded. By performing a large number of analyses with representative earthquake time histories, the probability of exceeding the load or damage level for the selected level of earthquake could be estimated. The properties of the structure could also be varied, to account for uncertainties affecting them; analyses of the structure with these varied properties would also be made. This would enable a complete weighted failure surface to be established over all possible variables in the analysis. The drawback to this approach is the large number of MDOF time-history analyses which would need to be performed.

The second approach would entail developing a response spectrum for the level of earthquake considered. This would be done by using an ensemble of time histories appropriate to the level of earthquake considered; the resulting spectrum would be smoothed to envelope the peak responses over a broad range of frequencies. This process is undertaken in the construction of all design response spectra.

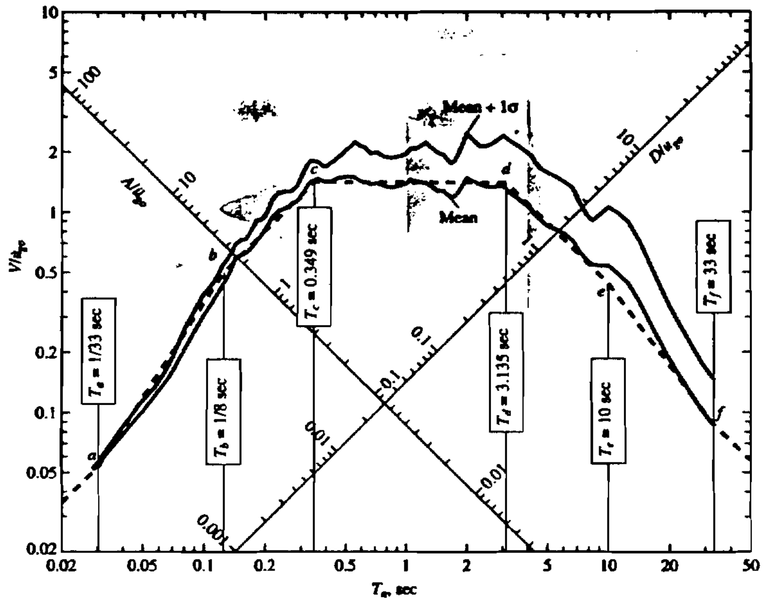

Figure 5.1: Uncertainty in Response Spectrum Ordinates

As seen in Figure 5.1, each response spectrum ordinate is a random variable. Typically, these ordinates possess coefficients of variation ranging from 40% to 80% (Bazzurro, Cornell, 1994). Most design spectra are usually established to envelope the median response within one standard deviation. Based on the distribution of the spectral ordinate, and the distributions of the modal properties, the distribution of the demand on the structure could be determined. This approach is far simpler than performing multiple time-history analyses; individual distributions of demand and capacity can be formulated and then used with reliability theory to determine probabilities of failure. The drawback to this approach is the difficulty in accounting for non-linear effects in MDOF systems as discussed in Section 3.0; however, for the purposes of strength-level assessments, the procedure should be quite adequate. Possible adaptations of the approach to yielding MDOF systems are discussed in Appendix A.

The statistical process by which reliability is evaluated using distributions of demand and capacity is described below. Defining a safety margin as:

$$M = \ln(R) - \ln(S)$$

where: R = resistance function of a structure or component

S = function describing demand on structure or component

The probability of failure may then be estimated from:

$$P_f = CDF\left(\frac{M - \mu_M}{\sigma_M}\right)$$

where: μ_M = mean value of the safety margin

σ_M = standard deviation of the safety margin

Assuming the demand on and capacity of the component may both be described by log-normal distributions, the exact reliability index may be solved explicitly. It is given by:

$$\beta = \frac{\mu_M}{\sigma_M}$$

where: $\mu_M = \ln\left(\frac{\mu_R}{\mu_S}\sqrt{\frac{1+V_S^2}{1+V_R^2}}\right)$

$$\sigma_M^2 = \ln\left(1+V_R^2\right) + \ln\left(1+V_S^2\right) - 2\ln\left(1+\rho_{RS}V_RV_S\right)$$

μ_R, σ_R = mean and coefficient of variation of resistance function

μ_S, σ_S = mean and coefficient of variation of demand function

ρ_{RS} = correlation between demand and resistance

The probability of failure is thus:

$$P_f = \Phi(-\beta)$$

where $\Phi(.)$ is the standard normal variate function.

The main task, then, is to determine appropriate log-normal distributions for the demand and capacity. This may be done by approximations such as the one described in the next section.

5.2 A Mean-Value First-Order Second-Moment Distribution Approximation for Earthquake Load Demands

The mean-value first-order second-moment (MVFOSM) reliability approximation is intended to allow estimation distributions of random variables so that reliability analysis under incomplete statistical information may be performed. This subject is discussed in Mansour (1985) and Mortazavi (1996); the essentials of the process will be described here.

Assuming the demand or capacity is a function F of random variables $(x_1, x_2, x_3 \ldots x_n)$, the following relationships may be used:

$$\mu_F \approx F(M_x)$$

$$\sigma_F^2 \approx \nabla F|_{M_x} \Sigma \nabla F^T|_{M_x}$$

where: $M_x = \begin{bmatrix} \mu_{x_1} & \mu_{x_2} & \cdots & \mu_{x_n} \end{bmatrix}$

$$\Sigma = \begin{bmatrix} \sigma_{x_1}^2 & \rho_{x_1 x_2}\sigma_{x_1}\sigma_{x_2} & \cdots & \rho_{x_1 x_n}\sigma_{x_1}\sigma_{x_n} \\ \rho_{x_2 x_1}\sigma_{x_2}\sigma_{x_1} & \sigma_{x_2}^2 & \cdots & \rho_{x_2 x_n}\sigma_{x_2}\sigma_{x_n} \\ \cdots & \cdots & \cdots & \cdots \\ \rho_{x_n x_1}\sigma_{x_n}\sigma_{x_1} & \rho_{x_n x_2}\sigma_{x_n}\sigma_{x_2} & \cdots & \sigma_{x_n}^2 \end{bmatrix}$$

$$\nabla F = \begin{bmatrix} \frac{\partial F}{\partial x_1} & \frac{\partial F}{\partial x_2} & \cdots & \frac{\partial F}{\partial x_n} \end{bmatrix}$$

To make use of these procedures, the means and standard deviations of the random variables x_i which make up the earthquake load (demand) and the strengths of the critical components (capacities) are needed, along with the relationship between these variables and the demand and capacity. Mortazavi (1996) has previously developed formulations for the means and standard deviations of deck and jacket bays strengths, and foundation pile axial and lateral strengths based on the means and uncertainties of their respective controlling random variables; these same formulations may be used for strength-level earthquake analysis. This reduces the present task to formulating the mean and uncertainty of the earthquake demand.

The mode-specific load demand calculated on a MDOF structure as found from a response spectrum is:

$$\mathbf{f}_n = \mathbf{s}_n A_n$$

where: $\mathbf{s}_n = \Gamma_n \mathbf{m} \phi_n$ = distribution of modal inertia forces

with: $\Gamma_n = L_n^h / M_n$ = modal participation factor

$$L_n^h = \sum_{j=1}^{N} m_j \phi_{jn}$$

$M_n = \sum_{j=1}^{N} m_j \phi_{jn}^2$ = generalized mass

$A_n = \omega_n^2 D_n$ = pseudo-acceleration; mA is equal to the peak value of the elastic resisting force for a SDOF system

and: n = mode index

j = DOF index

It was noted previously that inherent variability in ordinates D_n from a smoothed response spectrum is on the order of 40% to 80%. This variability may be further increased by uncertainties in the structure's natural periods and associated damping ratios. It is believed that this large uncertainty will dominate any others associated with the modal properties of the structure (Bazzurro, Cornell, 1994); hence, it is assumed uncertainty in the mass, stiffness and damping properties are small and may be neglected in so far as they affect the modal participation factor and mode shape. Therefore, the uncertainty in the modal forces is assumed to be equal to the uncertainty in the response spectrum ordinate. If it is desired to evaluate the uncertainty in the modal periods in order to assess the additional uncertainty of the response spectrum ordinate, the following relationship may be used to find the additional uncertainty:

$$\sigma_T^2 = \sigma_M^2 \left(\frac{\partial T}{\partial T_M} \right)^2 + \sigma_{K_x}^2 \left(\frac{\partial T}{\partial K_x} \right)^2 + \sigma_{K_\theta}^2 \left(\frac{\partial T}{\partial K_\theta} \right)^2$$

where T is the adjusted 1st mode period described in Section 3.0. It should be noted that the 1st mode period could have uncertainty approaching 50% or more if the foundation and mass uncertainties are all on the order of 30% or greater. However, if the uncertainty associated with D_n is 80%, the effective increase will only be on the order of 12%. If, however, D_n has been determined from an exact spectrum, the uncertainties in the structure's modal parameters will become more important.

This simplification, assuming that the total load uncertainty is equal to the uncertainty in the response spectrum ordinate, is only valid if the uncertainties associated with mass and stiffness are small. This may not be true for an offshore structure, when considering added mass effects and foundation stiffness and damping. It may be necessary to develop formulations including these uncertainties.

6.0 FUTURE WORK

The future effort devoted to the seismic analysis task will consist of the following:

Yielding Systems: Comparison of simplified analysis of yielding systems on flexible foundations with analysis results from detailed non-linear time-history studies should be made in order to assess the utility of the approaches discussed in Appendix A. Performing multiple analyses should provide data by which statistical trends between detailed and simplified analyses can be identified. A variety of systems (different heights and configurations) with different structural elements (elastic-plastic, stiffness and strength degrading) should be examined. Of key importance is the ability to identify the correct collapse mechanism, and then predict the demands associated with that mechanism. It must be determined if the collapse mechanism is strongly controlled by the imposed ground motion.

Assessing Load Path Integrity: Simplified means of assessing load path integrity need to be developed. The procedures examined in Appendix A assume that load paths within critical components remain intact. This may not necessarily be true, and may result in the formation of more complicated collapse mechanisms.

Torsion: Torsion response adds significant complication to evaluating seismic response. Mass and stiffness eccentricities may give rise to significant torsion modes. These modes may contribute significantly to the most likely collapse mechanisms.

Historical Verifications: These are essential to the calibration of any analysis process. As mentioned previously, recent assessments of offshore platforms indicate substantial differences between calculated mode shapes and periods and those determined by field testing. The areas of discrepancy must be identified and accounted for. Also, for the purposes of post-event inspection, it is desired to perform evaluations of real structures subjected to significant earthquakes, in order to assess the utility of using simple systems for damage identification.

Reliability Calibration: It is desirable to compare the results of using simple systems in making reliability assessments with current state-of-the-art methods. This will entail performing reliability assessments for non-linear systems using the time-history approach outlined in Section 5.0, and then comparing it to the results of simplified approaches. Also, it is necessary to assess the importance of variability in the mass, stiffness and damping properties on the response of these structures.

Damage Stiffness: The current stiffness formulations used within the ULSLEA program do not take into account reduction due to damage. This typically results in overestimation of load attraction for damaged members, and premature failure indications. Parameter studies in which various types of damage were experimented with, and then the effect on global capacity determined, would be very desirable.

General uncertainty and bias determination: Further literature surveys into biases and uncertainties affecting earthquake loads, foundation parameters, and added mass should be made.

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APPENDIX A: Response Spectrum Analysis Adaptations for the Evaluation of Non-Linear Systems

The initial focus of this research has been upon strength demands and strength capacities. However, as discussed in Section 2.0 it is not economically feasible to design based upon strength for all levels of earthquakes. In some extreme cases, damage will have to be tolerated. This entails allowing members in the structure to undergo inelastic deformation, so long as the ultimate performance criteria (usually collapse or emission of hazardous materials) are not exceeded. Hence, means of assessing inelastic response are needed; procedures for both the prediction of inelastic demands and the determination of inelastic capacities must be utilized.

A.1 Assessing Inelastic Demands

Today it is possible to make use of many very detailed structural analysis programs to perform non-linear time-history analysis of structures subjected to earthquakes. While these programs are of great use in the determination of response, there are several drawbacks. First of all, the amount of effort which must be expended in developing a suitable model which will capture behavior in a meaningful way is great; in addition, the actual analysis may place a large demand on computer capability. Also, the structure must be analyzed for a series of earthquake time histories, in order to ensure behavior is satisfactory over a range of earthquakes. The amount of effort needed to develop models for and perform multiple analyses of hundreds or even dozens of large structures would be prohibitive in terms of time and cost. Hence there has been a great emphasis over the past forty years to develop simplified procedures for assessing inelastic structural behavior under dynamic loading.

Most of the simplified procedures developed to date are based on response spectrum analysis. Two approaches to predicting response have been taken:

- Identifying similar trends in the peak responses of elastic and inelastic systems, and then attempting to develop correlating factors by which inelastic response can be predicted from the results of elastic response spectrum analysis.
- Attempting to determine the energy input to the structure during excitation, and then evaluating the mechanisms by which it can be dissipated (damping and/or inelastic deformation).

Of the two approaches, the former has received more attention over the years. There exists a large amount of material on the statistical observations of elastic, inelastic, and degrading systems; the reader is referred to Miranda and Bertero (1994), Riddell and Newmark (1979), Bazzurro and Cornell (1994) and Haviland, et al. (1976) for additional information. The main goal of these studies has been to develop factors which relate inelastic deformation demands to the forces calculated from a linear elastic response spectrum. A sample of measurements of inelastic deformation or ductility demand μ for SDOF systems of different period and allowable overload ratio (R_y, the inverse of f_y) are shown below.

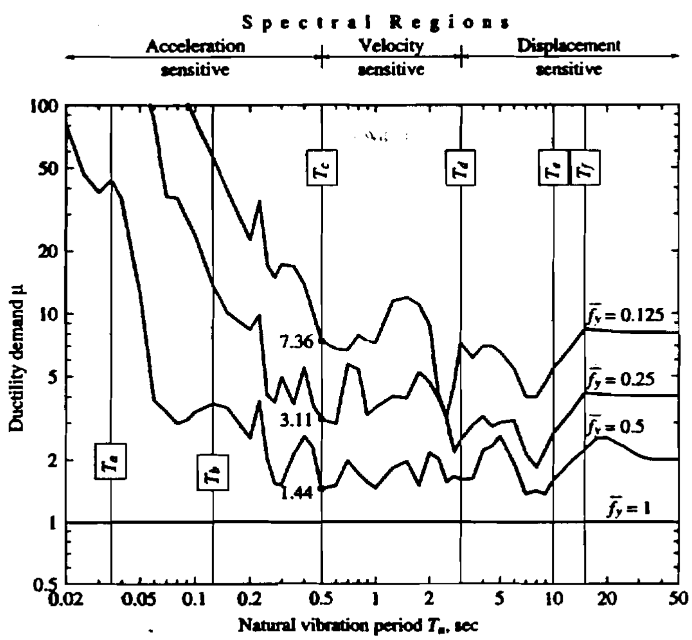

Figure A.1: Normalized Strength and Inelastic Displacements (Chopra, 1994)

In Figure A.1:

$$\bar{f}_y = \frac{f_y}{F}$$

$$\mu = \frac{u_{\max}}{u_y}$$

Obviously, there is much scatter when attempting to correlate the allowable overload ratio with the associated ductility demand for all but the longest periods. Based on repeated observations of SDOF systems, the following approximate relationships were proposed to correlate overload ratios with ductility demands (Newmark, Hall, 1982):

For the region between T_b and T_c:

$$u_{\max} = \frac{\mu u_{\max\text{ elastic}}}{\sqrt{2\mu - 1}}$$

$$\mu = \frac{R_y^2 - 1}{2}$$

For the region beyond T_c:

$$u_{\max} = u_{\max\text{ elastic}}$$

$$\mu = R_y$$

For the region below T_b, no approximations exist; for systems with even slight overload ratios, the resulting ductility demand can be enormous, as can be seen from Figure A.1. The UBC (1994) makes use of the relationship for the region beyond T_c within its seismic load estimating approach by allowing load reduction in proportion to R_w factors, which are taken as being equal to the expected allowable ductility of the structure. It should be noted that the relationships listed above are very approximate; there is much uncertainty associated with them.

The main weakness of most approaches relating overload and ductility demands for MDOF structures is the assumption is that the yielding is roughly distributed over the whole structure. For systems which possess more than three significant DOF, this can lead to unconservative results, with errors increasing as the number of DOF increases (Veletsos, 1969; Veletsos, Vann, 1971). In MDOF structures, yielding shifts vibration frequencies and changes mode shapes, which in turn will change modal participation factors and distributions of the inertial forces. In assuming the yielding is distributed over the structure, damage concentration at the locations of initial yielding is ignored, which may seriously underestimate the local ductility demands placed upon a structure (Chopra, 1995). However, for structures in which elastic response is concentrated in the first one or two modes, and possess fairly uniform stiffness and strength, these approximations can be adequate indicators of performance.

To overcome the limitations of the assumption of uniform yielding, some researchers have attempted to come up with procedures to account for damage concentration. Kennedy, et al. (1984) have proposed an adaptation to the ductility-modified response spectrum approach which is intended to account for localized structural non-linearity; based on successive estimates of the tangent stiffness for yielded elements, the modes and periods of the yielding structure are successively recalculated until the peak estimates of displacement between successive calculations are in good agreement. This procedure may have promise in improving estimates of ductility demand, but it has much uncertainty and is cumbersome to implement. Bazzurro and Cornell (1994) describe a procedure by which overload ratios may be correlated to specific ductility demands; however, this procedure requires successive time-history analyses in order to define the statistical mean relationship between the overload ratio and ductility factor. Miranda (1996) and Nassar and Krawinkler (1991) have attempted to define modifications to the system ductility factors to bound possible damage concentration in MDOF structures.

The other main approach to the problem has been based on balancing energy input to the system and energy absorbed and dissipated. The main difficulty is determining a measure by which the energy input and output mechanisms can be defined. Blume (1960) proposed a reserve energy method by which the energy-absorbing capabilities of different elements within a structure were compared to the approximate kinetic energy existing in the structure above the level of the element in question; the energy at each level is estimated from:

$$E_{x-1} = \sum_{i=x}^{n} \frac{1}{2} m_i V_1^2$$

where: m = individual masses above level

V_1 = spectral velocity of 1st mode

The displacement demand is established by equating this energy to the work done on the section in question. This approach was discussed briefly by Gates, et al. (1977) as a simplified alternative to estimating strain demands on structures. The approach attempts to conservatively estimate the amount of kinetic energy input to the structure by finding the kinetic energy of each mass, and then checking for overload in each level below the mass. This approach is extremely approximate, but is believed to be very conservative (Gates, et al., 1977).

In order to bound the ductility demand imposed upon elements within the structure, both the ductility-overload approach (assuming uniform yielding) and the reserve energy method (with damage concentration) could be used. However, additional work is needed in verifying the suitability of each approach to making damage predictions. Each approach contains great uncertainties and simplifying assumptions which must be understood prior to application.

As a sample application, a fixed-base model of the Southern California Test Structure was developed and analyzed by both time-history analysis and response spectrum analysis. Three simplified approaches to estimating ductility demands were applied based on the results of the response spectrum analyses:

- Estimating ductility demands through the consideration of elastic overload ratios, as discussed previously (Newmark, Hall, 1982).
- Estimating ductility demands at the most overloaded portion of the structure by considering them to be equal to the spectral displacement of the first mode.
- Using the Blume reserve energy technique.

The results of these approaches were then compared to ductility demands obtained using time-history analysis. The post-buckling behavior of the braces was assumed to be elastic-perfectly plastic. These comparisons were done for three time histories: Northridge (1994) Sylmar County Hospital, Northridge (1994) Newhall Fire Station, and Hanshin (1995) Miyagi Ken-Oki. The resulting estimated ductility demands for each level in the structure, along with the ductility demand determined from the time history analysis, are shown below in Figure A.2.

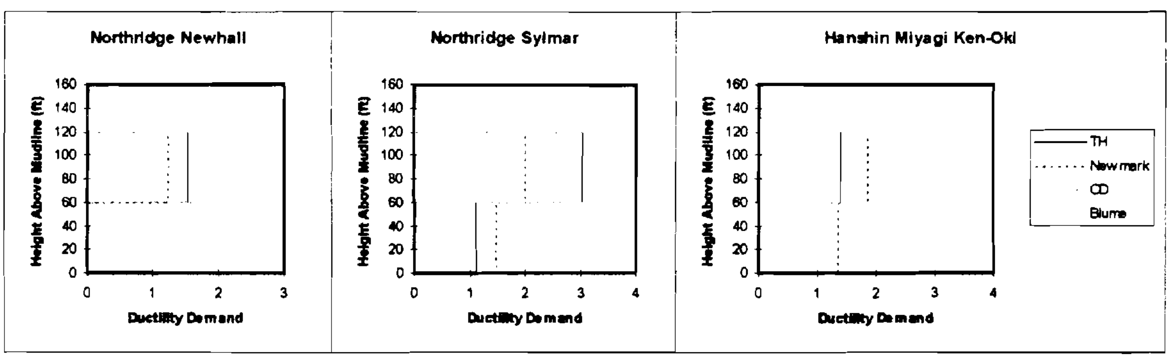

Figure A.2: Ductility Demands by Various Methods

Obviously, there is great scatter in the results of the three methods relative to the time-history results. Both the concentrated displacement approach and the reserve energy technique envelope the peak ductility demands for all three cases; however, they are quite conservative for the Hanshin Miyagi Ken-Oki time history. The Newmark-Hall approach tends to under-predict the demand for the Northridge time histories, but envelopes the Hanshin Miyagi Ken-Oki time history quite well. These results highlight the fact that the relationship between overload ratios from a linear elastic analyses and ductility demands are dependent on the type of excitation; for pulse-type loads like Northridge, Newmark-Hall appears inadequate, but for periodic excitation like Hanshin Newmark-Hall works well. Similarly, the concentrated displacement and reserve energy methods work well for pulse-type loads, but are extremely conservative for periodic loads. It must also be remembered that this example is a very narrow sample; there may be other cases for which all methods are inadequate.

A.2 Determining Capacity

On the capacity side, structural capacity must now be formulated in terms of strain and stability. Strain limits for components making up an offshore platform may be established through the consideration of test data. Various modifications to these values may be necessary to account for cyclic degradation of the material and strain rate effects (such as rapid loading for joints). This is a difficult task, as the capacity of the material may be governed by aspects of the applied load. However, several approximate relationships have been developed for braces and bending members by which good estimates of ductile capacity can be made with the effects of cycling already accounted for. A relationship for braces proposed by Astaneh (1996) is shown below:

Local buckling:

If $D/t \le \lambda_p$ then $\mu = 15$

If $\lambda_r \ge D/t > \lambda_p$ then $\mu = 15 - 14\left(\dfrac{D/t - \lambda_p}{\lambda_r - \lambda_p}\right)$

If $D/t > \lambda_r$ then $\mu = 1$

where: $\lambda_p = \dfrac{1{,}300}{F_y}$

$$\lambda_r = \frac{3{,}300}{F_y}$$

$$\mu = \frac{u_{\text{max member compression strain}}}{u_{\text{member compression yield strain}}}$$

Column buckling:

If $\lambda_c \leq 1$ then $\mu = 15$

If $1.5 \geq \lambda_c > 1$ then $\mu = 15 - 28(\lambda_c - 1)$

If $\lambda_c > 1.5$ then $\mu = 1$

where: $\lambda_c = \dfrac{kl}{\pi r}\sqrt{\dfrac{F_y}{E}}$

It should be noted that these proposed formulas contain significant uncertainties from testing procedures. Also, they are the result of monotonic load testing; to account for cyclic action, Astaneh (1996) has recommended reducing the maximum ductility achievable by a factor of three or four, and then scaling the relations in the intermediate regions between the maximum and minimum (unity) accordingly.

Applying this approach to the bracing members in the Southern California Test Structure, and using a reduction factor of two, member ductilities in the range of 4 to 5 were estimated for the 1st jacket bay diagonals; these estimates compare well to test data on these members (Zayas, et al., 1980B) which indicate cyclic ductilities of 5 or higher.

For bending members such as deck legs or piles, inelastic behavior is controlled by local buckling. The following limits are suggested by Astaneh (1996):

If $D/t \leq \lambda_p$ then $\mu \geq 6$

If $\lambda_r \geq D/t > \lambda_p$ then $\mu \geq 3$

If $D/t > \lambda_r$ then $\mu = \dfrac{M_p}{M_y}$

where: $\mu = \dfrac{\phi_{\text{max cross section rotation}}}{\phi_{\text{rotation at } M_y}}$

Maximum rotation ductilities are expected to be on the order of 15 for members meeting the compact section criteria.

Further work is needed to define stress-strain relationships for piles, as well has suitable strain limits. This is an area which has been the subject of much research, and will be examined further in the next phase of the project.

APPENDIX B: Spectral Accelerations for Deck-Mounted Equipment

Experience with industrial facilities and buildings subjected to intense earthquakes indicates that one of the most frequent sources of severe difficulties is associated with improperly tied down equipment, piping, storage tanks, and other similar facilities (Johnson, Bragagnolo, 1995). Recently, API RP 2A guidelines for design of offshore platforms has addressed this issue (API, 1993). As opposed to performing 3-D time-history analyses of the entire platform to intense earthquakes to generate deck response spectra, a simplified method to determine whether or not existing tie-downs onboard a platform are sufficient is desired.

A simplified engineering approach proposed by Biggs and Roesset (1970) to define earthquake floor accelerations for equipment, piping, and other facilities mounted on the decks of drilling and production platforms is discussed in this section. The simplified approach is based on relatively simple calculations that would require as input the platform ground motion elastic response spectra, the platform primary response periods and mode shapes, and the estimated weights and periods of the equipment of concern. The formulation of this approach is founded on the results from comparable developments of floor spectra guidelines for nuclear power plants, buildings, and refinery vessels and piping. The material in this Appendix is excerpted from Bowen, Bea (1995).

B.1 Review of Current Approaches

This section reviews some of the current approaches to the problem of simplified deck response spectra generation. These approaches are listed below:

Tan and Lung (1992)

Der Kiureghian and Igusa (1985)

Newmark, Nakhata, and Hall (1973)

Ruzika and Robinson (1980)

Biggs and Roesset (1970)

The following selection criteria should be considered when choosing an approach:

1. Accuracy: The method must produce results that represent the actual structure and its behavior. The results do not have to be 'exact', but suitable for the intended purposes. There is a degree of uncertainty in any engineering process, and the output from this method must be within acceptable limits of accuracy.
2. Consistency: The approach should be able to produce similar results for similar problems when used by different engineers. The method must be suitable for application to different types of equipment, piping systems and storage-processing vessels located on offshore platforms.
3. Input: The necessary information must be readily available for use in the method. If the method is itself simple, but obtaining the necessary data to implement the method is difficult, then the method's effectiveness is significantly reduced.
4. Output: The output must be readily understood by the practicing engineer. The output must be unambiguous. If the output is too limiting, simple, or complex, the effectiveness of the method is reduced.
5. Compatibility: The method should be readily integrated into the engineering practices incorporated into the earthquake engineering guidelines of API RP 2A. Desirably, the method should require simple calculations.
6. Verification: The results from the method should be readily verified using first-principles, experimental results, results from other analytical models, and intuition. Intuitive verification refers to the fact that the results must "makes sense". If the

method is so complicated that the engineer does not have a feel for the results, serious problems will arise.

7. Documentation: A procedure must be able to be understood by an engineer. If very complex theory and mathematics are required, the method will not be accepted by practicing engineers. The written procedures must be clear, sufficiently detailed, and correct. The documentation must represent effective and efficient transmission of information on how to use the procedure, how not to use the procedure (its limitations), how to provide correct input information, and how to interpret the results.

Generally speaking, all the methods reviewed were both accurate and addressed the output required. Many of the approaches were sophisticated and ingenious. Verification, compatibility, and procedure documentation were a distinct obstacle to many of the methods. The methods reviewed are summarized in the following paragraphs. Table B.1 summarizes the ratings of the alternative formulations (attributes are keyed to numbers above).

METHOD	ATTRIBUTE						
	1	2	3	4	5	6	7
Tan & Lung, 1991	High	High	High	High	Medium	Low	Low
Der Kiureghian & Igusa, 1985	High	High	High	High	High	Low	Low
Newmark, Nakhata, & Hall, 1973	High	Medium	High	High	High	High	Medium
Ruzicka & Robinson 1980	High	Medium	High	High	Medium	Low	Low
Biggs & Roesset, 1970	High	High	High	High	High	High	High

Table B.1 - Evaluation of Floor Response Spectra Approaches

1. Tan and Lung (1992) - Accuracy, input and output attributes of this approach were evaluated to be high. The method uses a response spectrum, and accounts for

interaction of the equipment masses and structure masses, and also different damping of the equipment and structure. Analytical verification of the method was judged to be adequate. The solution foundation is based on the Laplace transform of the equation of motion, and the inversion of Green's theorem by residue theory. Clearly, this is not something that the average engineer could have a physical feel for. Additionally, the basic method uses multiple supports of equipment; at the base and the ceiling. However, it can be adapted for single base support.

2. Der Kiureghian and Igusa (1985) - This method has high accuracy (closed form solution), accounting for mass interaction and different damping characteristics. The method is based on the use of a composite system of the equipment and structure in the equation of motion. Analytical verification of the method was judged to be adequate. The method uses perturbation methods to solve the composite system. Although only elementary techniques of perturbation are used, the method involves very sophisticated and advanced dynamic analysis. The method also uses the Laplace transform to solve the equation of motion, and plots frequencies on the complex plane. The procedure is difficult to grasp and not easily verified. Implementation of the procedure would require a sophisticated computer program.

3. Newmark, Nakhata, and Hall (1973) - This is one of the better methods, and a close second to the one chosen. The significant input to the equipment excitation is a series of harmonic excitations with frequencies equal to that of the structure. The method partitions the coefficient matrices of a component system, then limits the equipment to a single point of attachment to simplify the equation of motion. The forcing function is then simply a function of the equipment stiffness and support displacement. A principal deficiency lies in the method used to combine modes: it is the absolute sum of the modal contributions. Additionally, the method restricts the input to having only one of the equipment and structure frequencies being equal. This severely limits the generality of the approach.

4. Ruzika and Robinson (1980) - This method was evaluated to have high attributes of accuracy, input and output. However, the method was evaluated to be a much less

consistent method due to the fact that it *only* concerns itself with tuned systems. The method is an asymptotic procedure that assumes small structure to equipment ratios for mass and stiffness, which may or may not be the case. The procedure uses the convolution integral, Fourier transform and frequency domain analysis to simplify the exact solution. The method is complex and would not be easily verified or utilized as a simplified procedure.

5. Biggs and Roesset (1970) - This method that was selected for implementation. Accuracy of the theoretical basis is improved by utilizing empirical results. The method allows for multiple degree of freedom structures, equipment, and different damping characteristics of the two. Both input and output are simple, and no time history analysis is needed. Only the significant mode shapes and periods of the structure and equipment are required. The procedure is uncomplicated, and may be performed with only a calculator. The theoretical basis is very intuitive, and analytical and experimental verification is elementary. The main drawback is that the method does not account for mass interaction, which when the natural frequencies of the equipment and the structure are close (referred to as tuning in the literature), may be significant. However, as the ratio of the mass of the equipment to the mass of the structure gets very small, the effect becomes much less a factor. For the majority of offshore applications, this would be the case. Furthermore, the effect of mass interaction makes the assumption of no interaction more conservative. In essence, the worst case scenario of significant interaction would result in an over strength design. With the inherent uncertainty in seismic analysis, this was not considered a major drawback.

In summary, the method selected (Biggs and Roesset, 1970) as a basis for development of a guideline to define platform topsides earthquake floor spectra has the following advantages and disadvantages:

Advantages : The method is relatively simple, and fast. Only the response spectrum and the dynamic characteristics (mode shapes, periods, and damping ratios) of the structure and the equipment are required. No time history analysis is needed. The procedure can be applied to multiple-degrees-of-freedom (MDOF) pieces of equipment or piping. The method allows for different damping ratios for the equipment and the structure. Advanced dynamic analysis knowledge is not required. Even though interaction of topsides and platform masses is not considered, as the ratio of mass of the equipment to the mass of the structure approaches zero, the effects become negligible. This is generally the case for offshore platforms. Neglecting the interaction effects tends to make the design more conservative.

Disadvantages: The method does not account for interaction of the platform and topsides masses. The mass of the equipment is assumed to be light enough compared to the mass of the structure to ignore interaction effects. Thus, the dynamic characteristics of the structure remain the same after mounting equipment. This interaction may or may not be significant. However, as stated above, ignoring interaction makes a more conservative design. The method assumes lumped mass systems. Theoretical results are calibrated to more closely match empirical results.

B2. PROCEDURE

The principal modification developed during this study is the equipment acceleration magnification ratio diagram. This diagram has been based on synthesis of the analytical approaches developed by Biggs and Roessett (1970) and calibrated with additional results from analyses of offshore platforms subjected to earthquake time histories (Bowen, Bea, 1995). The acceleration magnification ratio diagram represents a mean result. Based on the time history results available to this study, at a given period ratio

(ratio of equipment period, T_e, to structure period, T_s) the coefficient of variation of the acceleration magnification ratio is estimated to range from 10% to 15%.

The magnification ratio diagram proposed is based on a structure damping ratio of 5% and an equipment damping ratio of 2%. As appropriate, other damping ratios can be used to develop other magnification ratio diagrams. The developments are based on linear elastic response of the platform and topsides and are applicable to API Strength Level Earthquake (SLE) conditions.

There are two limiting cases of equipment response that are important to understand. The first is the case of rigid equipment in which the equipment is very stiff compared with the supporting structure. An example might be a horizontal separator skid that is mounted on the platform deck. The equipment simply must move in the same manner as its support. The motion and the maximum acceleration of the equipment mass, A_e, must be the same as that of the supporting point on the structure, A_s. Thus $A_e = A_s$.

The second limiting case is that of very flexible equipment. An example might be a flare boom or flare stack mounted on the platform deck. The period of the equipment, T_e, is much greater than that of the supporting structure, T_s. The internal distortion of the structure is relatively unimportant and the equipment behaves as though it was supported directly on the ground. In this case, the maximum acceleration of the equipment is equal to the maximum acceleration of the ground.

Between these two limiting cases, there is interaction between the equipment and the structure. The structure behaves as a frequency filter, developing harmonic components with frequencies equal to the modal frequencies of the structure. If the equipment has a natural frequency close to one of these harmonic components, the motion can be amplified. Near the point of resonance ($T_e \approx T_s$), the maximum acceleration of the equipment can be several times that of the supporting structure. The amplification (A_e /

A_s) will be proportional to the number of cycles of motion, N (for low damping $A_e / A_s \approx N\pi$). Given a sufficient number of cycles (e.g. $N \geq 3$), the amplification is limited by damping ($A_e / A_s \approx 0.5\xi$). Based on the results of time history analyses of structures with mounted equipment (Bowen, Bea, 1993), the relationship between accelerations for structure and equipment may be expressed as shown in Figure B.1.

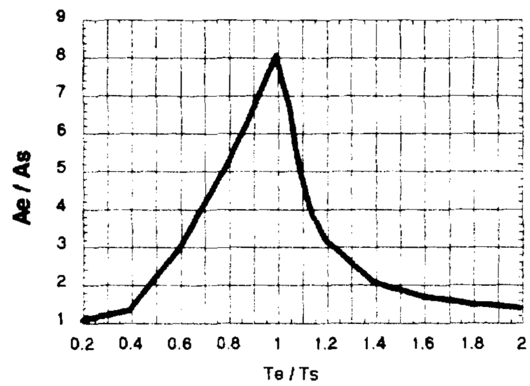

Figure B.1: Equipment/Structure Amplification Ratios

The procedure to find the appropriate acceleration for use in determining equipment tie-down forces is organized into five steps:

1. Obtain the acceleration at the DOF x corresponding to the point of equipment support for each structure vibration mode i:

$$\ddot{u}_{xi} = \Gamma_i \phi_{xi} SA_i$$

2. Obtain the spectral accelerations for the equipment modes j:

$$\ddot{u}_j = SA_j$$

3. If the ratio of T_{ej} / T_{si} is less than 1.25, modify the structure's acceleration $\ddot{u}_{xi}$ at the DOF of attachment by the ratio of A_{ej} / A_{si} taken from Figure B.1 and assign to $\ddot{u}_{ij}'$:

$$\ddot{u}_{ij}' = \left(\frac{A_{ej}}{A_{si}}\right)\ddot{u}_{xi}$$

Otherwise, modify the equipment mode's spectral acceleration by A_{ej} / A_{si} , and assign to $\ddot{u}_{ij}''$:

$$\ddot{u}_{ij}'' = \left(\frac{A_{ej}}{A_{si}}\right)SA_j$$

4. Perform the above tasks for each mode of the structure. When done, combine the resulting accelerations according to the following to get the equipment modal acceleration:

$$\ddot{u}_j = \sqrt{\sum_{i \text{ over all } \ddot{u}'} \left(\ddot{u}_{ij}'\right)^2 + \frac{\sum_{i \text{ over all } \ddot{u}''} \left(\Gamma_i \phi_{xi} \ddot{u}_{ij}''\right)^2}{\sum_{i \text{ overall all structure modes}} \left(\Gamma_i \phi_{xi}\right)^2}}$$

5. Repeat the above tasks for each equipment mode. This will provide spectral accelerations for all equipment modes, after which forces can be determined using modal analysis procedures.

APPENDIX C: Modified UBC Approach

As an alternative to rigorously evaluating the vibration properties of a platform using modal analysis and then applying the response spectrum approach, there exist several semi-empirical earthquake-demand estimating approaches such as the one contained within the Uniform Building Code (1994). These approaches are based upon the study of general trends of structural response to earthquakes, and are intended to allow for the development of forces with which a structural design can be started.

The UBC approach for horizontal forces assumes the structure in question has no great stiffness discontinuities, and that higher mode effects will decrease rapidly in significance. A total approximate base shear is estimated, and then distributed over the height of the structure in proportion with the mass at each level. In addition, a concentrated force is applied at the top to ensure that forces from higher modes will not be neglected in the upper portions of the structure.

To evaluate the utility of this approach in estimating earthquake demands for offshore structures, the basic force estimating procedure has been adapted for use with the API response spectrum. Base shear (immediately above the foundation) is estimated from:

$$V = SA_{T_1} W$$

where: SA_{T_1} = Pseudo-acceleration from response spectrum for 1st natural period

W = Total mass of the structure, not including foundation

The 1st natural period determined by first estimating the period for a fixed-base condition (by use of empirical formulas or through application of Rayleigh's method), and then modifying this period in accordance with the period-lengthening factors discussed in Section 3.3.2.

The forces distributed at the various levels in the structure are then determined in accordance with:

$$F_x = \frac{(V - F_t) w_x h_x}{\sum_{i=1}^{n} w_i h_i}$$

where: F_t = $0.07VT_1$ = concentrated force at top of structure, in addition to F_x

w = mass at level

h = height of level

Forces on the foundation of the structure are then approximated by the procedure described in Section 3.3.2. A SRSS combination of base shear and foundation forces is judged to be appropriate in estimating the total strength demand on the foundation.

As can be seen from the results of the case studies in Section 4.0, this modified UBC approach provides in most cases an excellent approximation to demands estimated by applying detailed modal analysis and then using the NRL-SRSS combination rule. There are concerns, however, as to how the procedure may err when stiffness discontinuities exist within a structure, as in the case of a platform with an unbraced deck bay.

To study this effect, a 2-D fixed-base model of the Southern California Test Structure was developed and analyzed with and without bracing in the deck bay. Shear demands for both the braced and unbraced case have been calculated using the modified UBC approach and modal analysis together with a response spectrum. These results may be seen in Figure C.1. For this case, there is little change in the overall load demand on the structure when the braces are removed; this would indicate that the structural discontinuity does not affect the overall response characteristics of the structure to any significant degree. This assumption must be tested further, however, to determine what limitations, if any, exist to the application of this modified UBC approach to offshore platforms. Also, the use of code forces for extremely tall structures, such as platforms sited in water depths exceeding 500 ft, should be evaluated and compared against more detailed analyses, in order to study the relative significance of higher mode effects on response. Research of this nature has been performed before for structures with fairly uniform mass and stiffness along their height; it would be interesting to see how the response of offshore platforms compares with these previous studies.

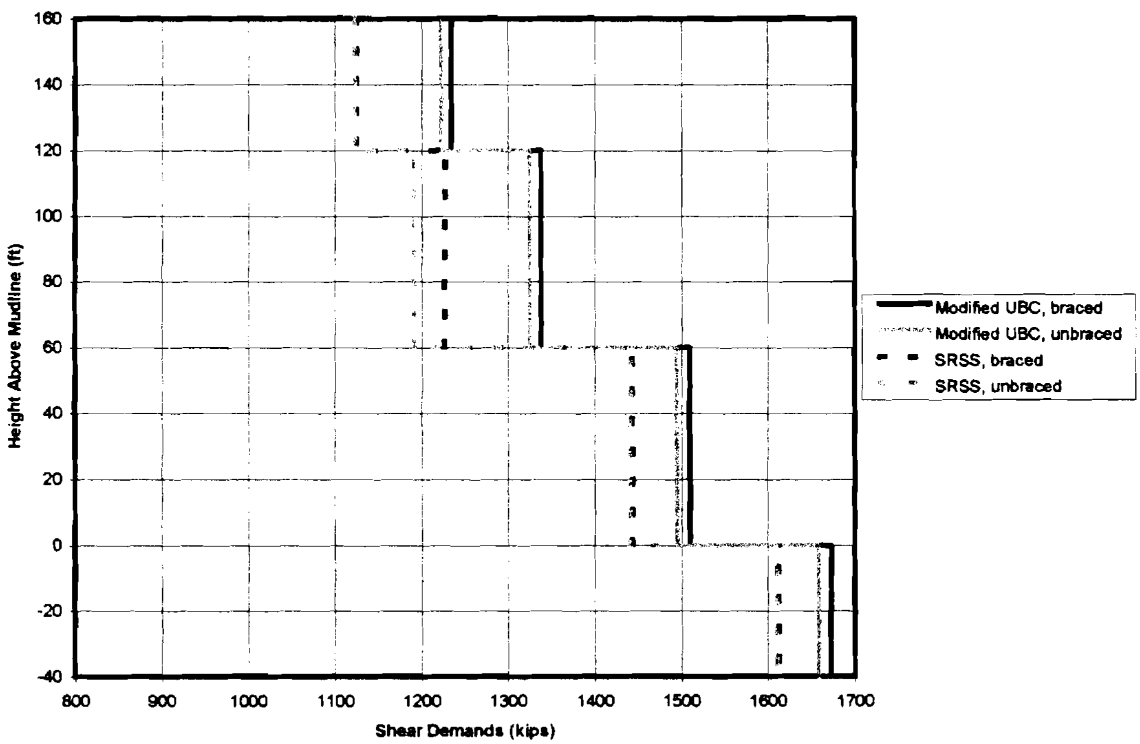

Figure C.1: Comparing Shear Demands for Braced and Unbraced Deck Bay, Southern California Test Structure conditions. This has important implications for soils which may undergo stiffness reduction from cyclic loading; the foundation may soften, changing the response characteristics of the platform significantly.

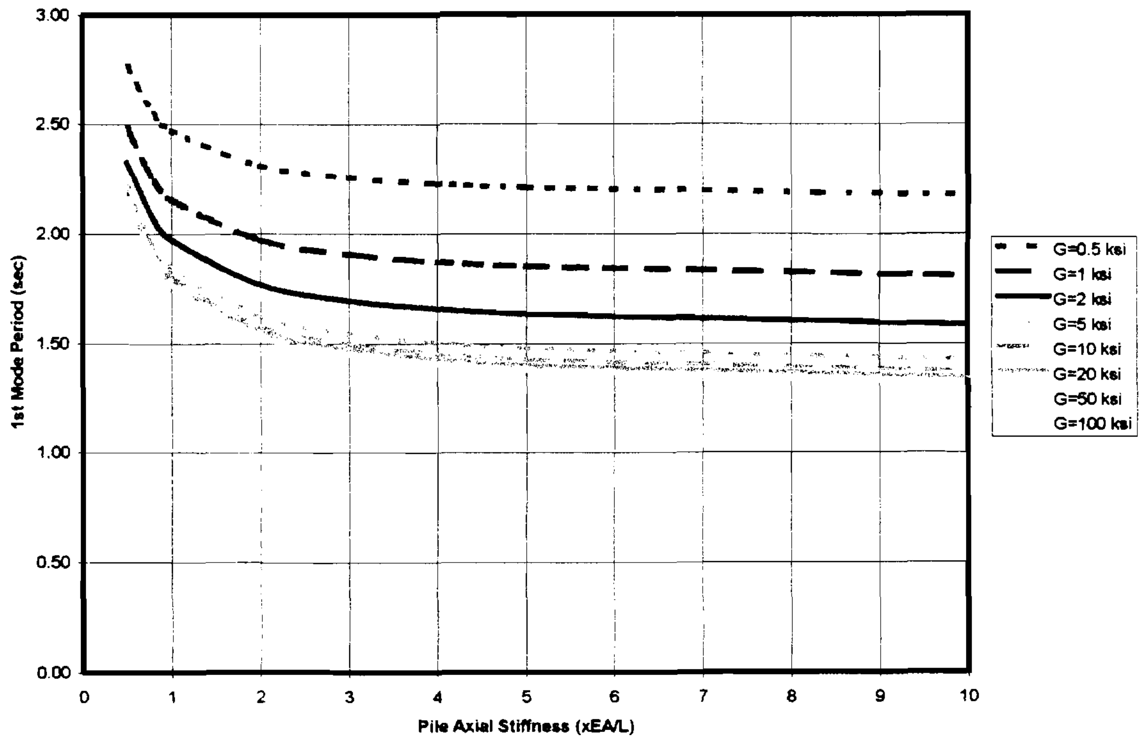

Figure D.2: Variation in T_1 with Changing Properties, Penzien Model

Figure D.3 shows the variation in 1st natural period for the end-on direction of Platform H, but in this case using foundation stiffnesses developed from Dobry (1980). Again, the 1st mode period is extremely sensitive to changes in the soil when it is very flexible. Two curves are shown, one for the case of weak jacket-pile connection stiffness (3EI/L, for the jacket leg and pile section above the mudline in the bottom jacket bay), and one for rigid jacket-pile connection stiffness. The effects of connection stiffness for this structure are small. It should be noted that for similar soil conditions, both approaches give similar results, indicating both models capture the essentials of pile-soil stiffness.

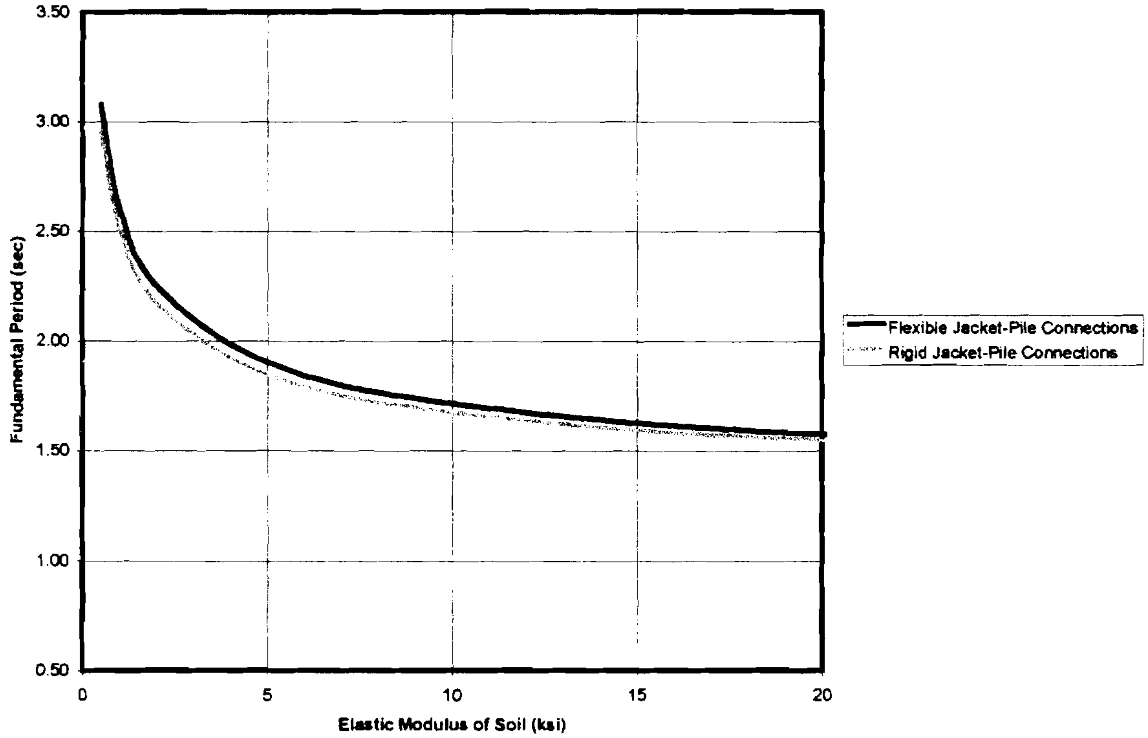

Figure D.3: Variation in 1st Natural Period with Changing Soil Properties, Dobry Model

APPENDIX E: Modal Analysis Through Solution of the Standard Eigenvalue Problem

The approach utilized within the ULSLEA program to find mode shapes and periods for horizontal response is based upon the conversion of the problem $\mathbf{k}\phi_n = \omega_n^2 \mathbf{m}\phi_n$ to standard form, $\mathbf{A}\phi_n = \lambda_n \phi_n$, and then solving for the eigenvectors and eigenvalues through iteration. This process has attracted much attention in engineering; readers desiring additional information are referred to Timoshenko, Young and Weaver (1974) and to Parlett (1980). The essential steps of the iteration process are listed below:

1. Start with an arbitrary trial vector, ϕ_n, and solve $\mathbf{A}\phi_n = \mathbf{y}$.
2. Obtain an estimate of the associated eigenvalue, λ_n, by taking the ratio between components of ϕ_n and $\mathbf{y}$ having the same index; hence $\lambda_n \approx y_i / \phi_{ni}$.
3. Normalize $\mathbf{y}$ by y_i to get $\bar{\mathbf{y}}$. Check to see if all $\bar{y}_i = \phi_{ni}$; if this condition is met, ϕ_n is a valid eigenvector and λ_n the correct associated eigenvalue. If not, set $\phi_n = \bar{\mathbf{y}}$ and return to step 1.

This iteration process has the useful characteristic of always converging to the largest eigenvalue and associated eigenvector. The vibration problem $\mathbf{k}\phi_n = \omega_n^2 \mathbf{m}\phi_n$ can be transformed to the standard form by multiplying both sides by $\mathbf{k}^{-1}$ and dividing both sides by ω_n^2. Hence the rearranged problem is now of the form $\mathbf{A}\phi_n = \lambda_n \phi_n$, where $\mathbf{A} = \mathbf{k}^{-1}\mathbf{m}$ and $\lambda_n = 1/\omega_n^2$. The solution will converge to the largest value of λ_n, which conveniently coincides with the inverse of the square of the lowest natural frequency; this is of course the frequency associated with the first mode of vibration.

In order to obtain eigenvectors and eigenvalues associated with higher modes, it is necessary to ensure that successive eigenvectors are orthogonal to one another (the orthogonality condition, $\phi_i \mathbf{m}\phi_j = 0$, ensures that the work done by i^{th} mode inertia forces going through j^{th} mode displacements is zero). This may be accomplished by enforcing the orthogonality condition when determining successive eigenvectors. This process is shown in the following section.

With the eigenvector ϕ_1 determined, and with a trial vector ϕ_2 estimated, applying the orthogonality condition $\phi_1 \mathbf{m} \phi_2 = 0$ gives (assuming a diagonal mass matrix):

$$m_{11}\phi_{11}\phi_{21} + m_{22}\phi_{12}\phi_{22} + m_{33}\phi_{13}\phi_{23} + \ldots + m_{jj}\phi_{1j}\phi_{2j} = 0$$

Solving for ϕ_{21} (this is an arbitrary choice) gives:

$$\phi_{21} = -\frac{m_{22}\phi_{12}\phi_{22}}{m_{11}\phi_{11}} - \frac{m_{33}\phi_{13}\phi_{23}}{m_{11}\phi_{11}} - \ldots - \frac{m_{jj}\phi_{1j}\phi_{2j}}{m_{11}\phi_{11}}$$

Calculating ϕ_{21} using the above expression prior to using ϕ_2 as a trial vector ensures orthogonality between ϕ_2 and ϕ_1. This may be accomplished using the following matrix multiplication:

$$\phi_{2\,\text{trial}} = \mathbf{T}_{S1}\phi_2$$

$$\text{where: } \mathbf{T}_{S1} = \begin{bmatrix} 0 & -\frac{m_{22}\phi_{12}}{m_{11}\phi_{11}} & -\frac{m_{33}\phi_{13}}{m_{11}\phi_{11}} & \ldots & -\frac{m_{jj}\phi_{1j}}{m_{11}\phi_{11}} \\ 0 & 1 & 0 & \ldots & 0 \\ 0 & 0 & 1 & \ldots & 0 \\ \ldots & \ldots & \ldots & \ldots & \ldots \\ 0 & 0 & 0 & \ldots & 1 \end{bmatrix}$$

$\mathbf{T}_{S1}$ is referred to as a "sweeping" matrix, as it acts to sweep out or suppress the first mode characteristics and allow the second mode to be come dominant. As $\mathbf{T}_{S1}$ is used with each iteration of ϕ_2, it may be used to reformulate $\mathbf{A}$ according to $\mathbf{A}_{S1} = \mathbf{A}\mathbf{T}_{S1}$, and then operate directly on the reformulated matrix.

Higher modes may be determined by successive application of sweeping matrices. For example, a sweeping matrix $\mathbf{T}_{S2}$ for removing the dominance of ϕ_2 may be constructed, and then used together with $\mathbf{T}_{S1}$ to allow the third mode ϕ_3 to become dominant. This matrix $\mathbf{T}_{S2}$ would be constructed by using the fact that $\phi_1 \mathbf{m} \phi_3 = 0$ and $\phi_2 \mathbf{m} \phi_3 = 0$. This gives the following:

$$m_{11}\phi_{11}\phi_{31} + m_{22}\phi_{12}\phi_{32} + m_{33}\phi_{13}\phi_{33} + \ldots + m_{jj}\phi_{1j}\phi_{3j} = 0$$
$$m_{11}\phi_{21}\phi_{31} + m_{22}\phi_{22}\phi_{32} + m_{33}\phi_{23}\phi_{33} + \ldots + m_{jj}\phi_{2j}\phi_{3j} = 0$$

Using the first equation to find a relationship for ϕ_{31}, the following relationship can be developed for ϕ_{32}:

$$\phi_{32} = -\frac{m_{33}\left(\phi_{11}\phi_{23} - \phi_{21}\phi_{13}\right)}{m_{22}\left(\phi_{11}\phi_{22} - \phi_{21}\phi_{12}\right)} - \frac{m_{44}\left(\phi_{11}\phi_{24} - \phi_{21}\phi_{14}\right)}{m_{22}\left(\phi_{11}\phi_{22} - \phi_{21}\phi_{12}\right)} - \ldots - \frac{m_{jj}\left(\phi_{11}\phi_{2j} - \phi_{21}\phi_{1j}\right)}{m_{22}\left(\phi_{11}\phi_{22} - \phi_{21}\phi_{12}\right)}$$

The sweeping matrix is thus:

$$\mathbf{T}_{S2} = \begin{bmatrix} 1 & 0 & 0 & \ldots & 0 \\ 0 & 0 & -\dfrac{m_{33}\left(\phi_{11}\phi_{23} - \phi_{21}\phi_{13}\right)}{m_{22}\left(\phi_{11}\phi_{22} - \phi_{21}\phi_{12}\right)} & \ldots & -\dfrac{m_{jj}\left(\phi_{11}\phi_{2j} - \phi_{21}\phi_{1j}\right)}{m_{22}\left(\phi_{11}\phi_{22} - \phi_{21}\phi_{12}\right)} \\ 0 & 0 & 1 & \ldots & 0 \\ \ldots & \ldots & \ldots & \ldots & \ldots \\ 0 & 0 & 0 & \ldots & 1 \end{bmatrix}$$

This is then used together with $\mathbf{T}_{S1}$ to reformulate $\mathbf{A}$ according to $\mathbf{A}_{S2} = \mathbf{A}\mathbf{T}_{S1}\mathbf{T}_{S2}$.